J. DVOŘÁK AND L. NOVÁK
(EDITORS)

SOIL CONSERVATION AND SILVICULTURE

DEVELOPMENT IN SOIL SCIENCE 23

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Soil conservation and silviculture

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CONTENTS

Preface 13
Chapter 1 **Soil conservation** (*J. Filip*) 15
1.1 Soil conservation and the danger of erosion 16
1.2 Conservation of farm and forest land 22

Chapter 2 **Erosion of the Soil** (*J. Dvořák*) 25
2.1 The types of erosion 25
2.1.1 Water erosion 25
2.1.2 Wind erosion 28
2.1.3 Glacial erosion 29
2.1.4 Snow erosion 29
2.1.5 Ground erosion 29
2.2 Manifestations of erosion 30
2.2.1 Manifestations of water erosion 30
2.2.1.1 Areal water erosion 30
2.2.1.2 Scouring erosion 33
2.2.1.3 Stream erosion 35
2.2.2 Manifestations of wind erosion 36
2.2.2.1 Deflation 36
2.2.2.2 Corrasion 36
2.3 Erosion intensity 37
2.3.1 Normal erosion 37
2.3.2 Accelerated erosion 37
2.3.3 Admissible intensity of erosion 37

Chapter 3 **Erosion Factors** (*J. Antal*) 39
3.1 Definition of erosion factors 39
3.1.1 Climatic conditions of the territory 39
3.1.2 Hydrological conditions in the territory 41
3.1.3 The topography of the territory 42
3.1.4 Soil conditions in the territory 45
3.1.5 Geological conditions in the territory 45
3.1.6 Vegetation in the territory 45
3.1.7 The economic and technical conditions in the territory 46
3.1.8 The socio-economic conditions 47

3.2 Evaluation of erosion factors 47
3.2.1 Runoff of precipitation water 47
3.2.1.1 Characteristics of design rain 48
3.2.1.2 Infiltration capacity of the soil 51
3.2.1.3 The rise of surface runoff of precipitation water 53
3.2.1.4 Characteristics of surface runoff 55
3.2.2 Erosion potential of water and wind 67
3.2.3 Erodibility of the soil 70
3.2.4 The effect of other erosion factors on erosion intensity 72
3.2.4.1 The effect of topographic conditions on soil erosion 72
3.2.4.2 The effect of vegetation on soil erosion . . 75
3.2.4.3 The effect of economic and technical conditions on soil erosion 77

Chapter 4 **Analysis of Erosion** (*J. Dvořák*) 81
4.1 Regularities of water erosion 81
4.1.1 Velocity of areal runoff 81
4.1.2 Tangential stress in areal runoff 84
4.1.3 Admissible slope length 84
4.2 Regularities of wind erosion (*M. Pasák*) 89
4.3 Modelling erosion processes (*F. Křovák*) 94
4.3.1 Wischmeir and Smith's empiric model of the erosion processes 94
4.3.2 Mathematical models of erosion processes 99

Chapter 5 **Erosion Control to Protect the Soil** 101
5.1 Anti-erosion site organisation (*J. Antal*) 102
5.1.1 Delimitation of the land fund 102
5.1.2 Woodland, grassland and cropland: distribution and location 105
5.1.3 The shape, size and position of fields 106
5.1.4 The communications network 109
5.2 Cultural operations for erosion control (*J. Antal*) . . . 111
5.2.1 Contour cultivation 111
5.2.2 Furrowing 113
5.2.3 Cultural practices for erosion control 115
5.3 Biological measures for erosion control (*J. Antal*) . . . 118

5.3.1 Soil conservation, rotation of crops 118
5.3.2 Strip cropping 118
5.3.3 Antierosion grassing 124
5.3.4 Forest shelter belts 124
5.3.5 Conservation afforestation 124
5.4 Technical measures of erosion control (*J. Filip*) 125
5.4.1 Terrain regulation 125
5.4.2 Terrace channels 126
5.4.3 Ditches 130
5.4.4 Check dams 134
5.4.5 Terraces 136
5.4.6 Erosion control reservoirs 143

Chapter 6 **Torrent Control** (*L. Novák*) 148
6.1 Torrent characteristics 148
6.2 Sediments 153
6.2.1 Properties of sediment grains 153
6.2.1.1 Sediment grain size 153
6.2.1.2 Sediment grain density and mass 154
6.2.1.3 Sediment grain shape 155
6.2.2 Properties of sediment mixtures 158
6.2.2.1 Location of sediment grains in the mix . . 158
6.2.2.2 Sediment mix composition 159
6.2.2.2.1 Sediment sampling 160
6.2.2.2.2 Sediment sample analysis and interpretation 161
6.2.3 Incipient sediment movement 166
6.2.3.1 The critical bed velocity method 167
6.2.3.2 The critical average channel velocity method 171
6.2.3.3 The critical shear stress method 172
6.2.4 Movement of sediment material 173
6.2.4.1 Bed load discharge 173
6.2.4.2 Sediment runoff 178
6.3 Principles of torrent control 182
6.3.1 Extent of torrent management 182
6.3.2 Methods of torrent control 183
6.3.2.1 Design discharge 183
6.3.2.2 Realignment 185
6.3.2.3 Adjustment of the bed slope 189
6.3.2.4 Designing the channel cross section 198

6.3.2.5 Channel lining 205
6.3.2.5.1 Bottom lining structures 206
6.3.2.5.2 Structures to line channel slopes . 208
6.3.2.6 Structures for torrent control 224

Chapter 7 **Gully Control** (*L. Novák*) 290
7.1 Gully erosion 290
7.2 Causes of gully erosion 290
7.2.1 Climatic conditions 291
7.2.2 Geomorphology 291
7.2.3 The soil and geological conditions 291
7.2.4 Vegetation 291
7.3 Gully forms 291
7.4 Gully stabilisation 293
7.4.1 Headcut control 294
7.4.2 Protecting the gully bottom 295
7.4.3 Securing the gully side slopes 296
7.5 Stabilisation of aged gully formations 298

Chapter 8 **Management of Land Endangered by Slippage** (*D. Húska*) 301
8.1 Causes of slope failure 301
8.1.1 Movement of surface deposits down the slope . . . 305
8.1.2 Stratified landslides 306
8.1.3 Liquefied earth 306
8.1.4 Liquid deposits 307
8.1.5 Landslides caused by sand washing 307
8.2 Methods of control and management of areas exposed to the danger of landslide 308
8.2.1 Field investigations 308
8.2.2 Interpreting the results of investigation 310
8.2.3 Securing the landslide territory 311
8.2.4 Special structural approaches to landslide stabilisation 315

Chapter 9 **Vegetation in the Landscape** 318
9.1 Forests in landscape (*J. Streďanský*) 318
9.1.1 Functions of forests 320
9.1.1.1 Water management function 320
9.1.1.2 Microclimatic and hygienic functions of forest 322
9.1.1.3 Soil-improvement function 323

9.1.1.4 Soil-conservation function 323
9.1.1.5 Recreational function 323
9.2 Fundamentals of the formation of vegetation (*J. Streďanský*) 323
9.2.1 Forest nursery practice 323
9.2.1.1 Seedling production 324
9.2.1.2 Production of plants 325
9.2.2 Afforestation 327
9.2.2.1 Soil preparation 327
9.2.2.2 Afforestation techniques 329
9.2.3 Treatment and protection of cultures 336
9.2.4 Filling the gaps and improving the stands 337
9.3 Tending of stands (*J. Streďanský*) 337
9.3.1 Techniques of forest stand tending 338
9.3.1.1 Tending of new growths and young crops 338
9.3.1.2 Tending of thickets 339
9.3.1.3 Tending of stands in small-pole and pole stages 339
9.3.1.4 Silvicultural treatment of maturing stands 340
9.4 Soil-conservation forest stands (*J. Streďanský*) 340
9.4.1 Grassing 341
9.4.2 Soil-conservation afforestation 342
9.4.2.1 Soil preparation 342
9.4.2.2 Choosing the species and the method of soil conservation afforestation 347
9.5 Riparian stands (*L. Novák*) 349
9.5.1 Riparian stands along water courses 349
9.5.1.1 Mission of riparian stands along water courses 349
9.5.1.2 Effect of riparian stands on the flow rate in a channel 351
9.5.1.3 Species composition of riparian stands . . 357
9.5.1.4 Spatial structure of riparian stands 360
9.5.1.5 Formation of riparian stands 364
9.5.1.6 Maintenance and regeneration of riparian stands 369
9.5.2 Riparian stands by resorvoirs 371
9.5.2.1 Using riparian stands to protect the banks of water reservoirs 372
9.6 Forest shelter belts (*L. Novák*) 378
9.6.1 Windbreaks 378

9.6.1.1 The favourable effect of windbreaks 378
9.6.1.2 The adverse effect of windbreaks 382
9.6.1.3 Formation of windbreaks 382
9.6.2 Infiltration belts 384
References 389
Subject Index 395

Preface

This book on conservation and improvement of the forest soil is intended for students at agricultural universities who specialise in soil improvement and for all readers who are interested in conservation of the forest environment.

The development of human society largely depends on the exploitation of the biosphere. Unfortunately, its resources cannot last forever: some may diminish, others may be irreversibly damaged or destroyed.

Land and water are among the key resources exploited by man. They are limited and irreplaceable. If degradation, pollution and wastage of soil and water continue, mankind's development may gradually slow down and stop.

Farming is intensifying as also are other branches of the economy, thereby aggravating the factors conducive to erosion.

It is the authors' hope that this book may contribute to the arduous effort to find solutions to these vital and complex problems of environmental conservation.

The authors offer their thanks to Associate Prof. Ing. V. Pasák, DrSc. and Associate Prof. Ing. A. Klopček, DrSc. for a detailed critical review of the manuscript.

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Jan Dvořák
Ladislav Novák

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CHAPTER 1

Soil Conservation

The soil is among the renewable natural riches of the Earth, but its renewal is extremely slow. The soil is essential for the production of food for man and indeed for all forms of life on dry land: it provides nutrients for plants which in turn are a source of food for animals and partly also for man; at the same time, plants release oxygen and water vapour which, as constituents of the air, are essential for life on the Earth. The soil also contributes to the conservation of another essential resource — water: if the soil has an undisturbed structure and a good vegetation cover, it can hold vast reserves of underground water.

Since the Neolithic, during which agriculture began and the climate already resembled present-day conditions, people have been using the soil to grow crops and raise animals (for as long as three to ten thousand years, depending on the geographical region): 97 to 99 percent of the food man consumes is provided in this way. However, where people exploited the land too intensively and removed the natural vegetation cover over excessively large areas (by turning forest into fields, by cutting trees for timber and fuelwood, by grazing their livestock in the forest), water and wind have carried the soil away, leaving nothing but bare rocks or dead desert in such areas. This process, in fact, destroyed certain advanced civilisations, e.g., in Babylonia, Syria and other areas. The absence of vegetation, especially forest, adversely affects the circulation of water — and where there is not enough water life can hardly survive: this applies to the majority of plant and animal species, including man.

Farm crop yields will decline even though the amount of fertile topsoil blown or washed away by wind or water may be very small: such slow erosion is in fact very insidious because it usually remains unnoticed for a long time. Erosion may cause further damage through the displacement of soil material and its deposition in a new area. As man is continuously increasing the intensity of soil exploitation in farming and forestry and destroying the soil cover by vast construction activity, there is an increasing call for soil conservation.

The conservation of farm and forest soil is becoming a necessity because with the increasing population of the world large areas of productive land are being offered up for construction and destroyed by advancing desertification and devastation. Soils are degraded by chemical pollution and also by increasing erosion: wind and water break the upper soil horizon, remove the material and deposit it somewhere else.

1.1 Soil conservation and the danger of erosion

The soil is an extremely complex living system. The soil itself consists of weathered parent rock (35 to 45% by volume), humus from decomposed biomass (0.5 to 10%), soil solution (biogenic inorganic nutrients and organics dissolved in water as ions or chelates) (15 to 35%), soil gas (15 to 35%), edaphon (microorganisms, worms, mites, insects, vertebrates and others) and higher plants, mainly their roots (up to 0.1%). Significant physical, chemical, biochemical and microbial processes take place in the soil (farmers and soil scientists are right in speaking about the *life* of agricultural and forest land). The permanent circulation of mass and energy exhibits dynamic changes, not only during the year but also over longer periods of time, thus maintaining a delicate biotic equilibrium. Healthy life inside the soil system has always played a key role in maintainig soil fertility and favourable soil properties for the cultivated plants.

The existing soils largely developed during the most recent period of geological time – the Quaternary: it began about 2 million years ago and included 5 cold periods, called glacial epochs, with average annual temperatures in Central Europe of between about −2 and −3 °C. These alternated with interglacial periods during which the average annual temperature was about 12 °C (for comparison, in the current epoch the average annual temperature in this region is 8 °C).

During the glacial periods the climate was cool to cold and comparatively dry, the type of vegetation was tundra or arctic steppe and winds blew steadily from the continental glaciers. The dust of weathered rocks was carried away by the wind by distances of hundreds to thousands of kilometres to the lowlands and foothills to produce layers of loess and eolithic sands up to several dozen metres thick. During the interglacial periods, which were warm and humid, the blown soils were overgrown by rank grass or forest vegetation which produced a humus layer on the soil surface. The glaciers thawed and raised the water level in the oceans by up to 120 m: the water then flooded the lowlands. This whole cycle has kept repeating over long periods of time. Deposition of material transported by the water from the thawing glaciers continued forming deposits of loam, sand and other materials in the lowlands during the Holocene. The formation of alluvial soils is still under way at the present time.

Like everything in the Universe, the soil keeps on developing. Contradictory tendencies are involved in the soil-formation process: synthesis and breakdown of mineral and organic substances, moistening and drying of the soil, warming and cooling, oxidation and reduction, etc. This is why we say that the soil is a separate natural entity living its own life and having its own transformation and transportation systems (JURČA, 1984). The complexity of the role played by the biotic factors involved in soil genesis is best characterised by the fact that within the chain of interdependent soil-forming factors they may act as causes in some cases and as consequences in others. The circumstances that reduce the multiform biological activity of the soil (erosion, use of chemicals, climatic cooling) slow down the soil's development; with a severe exposure to such factors, the soil development process may even be stopped or degraded.

Excessive erosion is currently the most serious threat to soil fertility. It is caused by the overexploitation of fields and pastures: people seek to produce as much

food as possible to feed the increasing world population and to support the increasing standard of living in some countries. Forests are also being exploited too intensively, irrespective of the worsening water balance and consequent intensifying of erosion.

Looking back into history, people living in different geographical regions developed many traditional agricultural systems; for example, the rotation of crops in the temperate zone, the fallow system in the semiarid and tropical areas, and terraced fields in some mountain areas. These systems, developing over thousands of years, had maintained their environmental stability until the middle of the 20th century, when the world's population was 2.5 billion. Today, with a population of 5 billion, such traditional systems can neither produce enough food nor control the environmental consequences.

A new farming method, large-scale agricultural production on an industrial basis, has developed in the areas of the temperate zone since the mid-20th century. It is a very productive system: one person permanently engaged in such farming is able to support as many as 20 to 50 people outside agriculture (this ratio used to be 1 : 1, later 1 : 3 – MEZŘICKÝ et al., 1986). This high performance of large-scale industrialised agriculture has contributed to eliminating hunger in spite of the unprecedented growth of the population; however, on the other hand, it has disturbed the dynamic balance of the world of Nature.

The original farming system was more or less stable, requiring comparatively small material and energy inputs and working within a more or less closed cycle (almost no waste was generated). In the industrialised system, on the other hand, the energy and material inputs (highly selected seeds and planting material, commercial fertilisers, biocides and other chemicals, commercial feeds, means of farm mechanisation, single-purpose farm machines, transport mechanisms, fuels, etc.) have exceeded the capacity of the internal cyclic metabolism, because wastes (residues of agricultural chemicals, silage juices, wastes from larger livestock herds, packagings, emissions polluting both water and the landscape) are generated at all stages of the production processes. In addition, the structure of the fields has been reorganised and their size has been increased to suit the high labour productivity and the high intensity of farming; all this is conducive to an enormous increase in soil erosion.

Also in the forestry areas of the temperate zone, the intensity of exploitation has increased considerably. This increased intensity combines with other harmful factors such as forest species structures not suitable to the local conditions, dense transport networks and, the most serious factor, the withering of forest as a consequence of acid rains in areas exposed to the greatest air pollution (central Europe, northern America). In Europe the area of damaged forests in 1984 was estimated at 5 million hectares (PEŘINA et al., 1987). As the forests are destroyed, more rain and snow water is lost through increased surface runoff which leads to increased erosion, to a reduction of underground water reserves and to higher peak flow rates in the water courses.

In the large semiarid regions where there is not enough moisture for uninterrupted utilization of the fields, and in the tropics where the soil is poor in nutrients, farmers developed the fallowing system of field management: fields are left to lie fallow for some time to gather moisture or new nutrients. In the semiarid regions

the soil is left uncultivated every other year to gain sufficient moisture for the next crop. In the tropical regions the deforested soil can be farmed for 2 to 4 years until the available nutrients of the soils are exhausted and the land is no longer able to give sufficient yields; vigorous wild vegetation is then left to quickly overgrow the plot. Some 20 to 25 years later the plot will again have enough nutrients to be changed back into a field.

The rapid growth of population in the semiarid areas and, sometimes, also the effort to make immediate profit and thereby repay the state debt rapidly in some countries force farmers to reduce the area left fallow: this aggravates wind erosion, gives rise to dust storms and leads to continuous reduction of soil fertility. In the tropics the traditional cycle is, for the same reasons, reduced to 5 years (Nigeria) and even to 2 years (Thailand). Even despite the application of commercial fertilisers, introduction of high-performance varieties and use of irrigation, the crop yields are not higher than they were in the 1960's.

In the tropics, new farm land is also obtained through the conversion of the rain forest and the tree savanna. However, cutting these forests has far-reaching and disastrous consequences: in the warm and humid environment of the rain forest, consisting of nothing but trees 50−65 m tall and lianas, all organic matter is liable to rapid and intensive decomposition and rocks easily yield to weathering. Hence, the soil is poor in nutrients which remain only in the leaves and the wood: in the deforested areas the nutrients disappear together with the upper layer of the soil. For example, the runoff rate from a forest on a 4° slope in Kenya at an altitude of 1500 to 2000 m above sea level is 60 litres per second per hectare but after felling it increases to 2.7 cubic metres per second per hectare (HADAČ et al., 1983).

As the annual population growth in the tropical regions (with minor exceptions) ranges between 2 and 3.5%, the only chance for this population to survive is to turn the forest into fields. This is the reason why as much as 7.5 million ha (0.61%) of rain forest and 3.81 million ha (0.58%) of tree and shrub savannas disappears from the surface of the Earth every year (POSTEL, 1984). This is so in spite of the fact that of the amount of oxygen that world forests release into the atmosphere the major part comes from the rain forests. This causes apprehension and so do the changes in the runoff and evaporation conditions in the large deforested areas. Deforestation also increases the prices of wood which is, in these regions, the only source of fuel and an important building material.

In the hilly regions of some countries, including China, Japan, Nepal, Indonesia and the Andean countries of southern America, the construction of terraces enables growing crops even on very steep hillsides − this has been so since the very early stages of the history of mankind. The old civilisations needed hundreds of years to build the ingenious systems of terraces. Nevertheless, even in these areas the increasing population (e.g., in China there is 0.1 ha of agricultural land per capita of population and in Japan 0.06 ha) forces farmers to build new terraces on the hillsides much more rapidly than ever before, not allowing the new terraces to stabilise. Terraces hastily built on the upper parts of the slopes are liable to rapid degradation and cause disastrous landslides which destroy whole villages (BROWN et al., 1984).

Erosion of the soil has considerably increased in all the existing world agricultural systems as well as in forests. According to S. Judson's estimate (in BROWN et al., 1984), the amount of soil displaced every year on the whole surface of the Earth has increased from 9 billion tonnes before the rise of agriculture to as much as 24 billion tonnes today. In this way, man has become a significant geological factor in the planetary metabolism whose activity has undesirable environmental consequences.

Unfortunately, the available information on soil erosion on the Earth is not accurate, so its extent can only be estimated on the basis of soil surveys and soil inventories in some countries; particularly valuable information can be obtained from the data on the amounts of material transported by the world's largest rivers. For example, the Huang He in China carries away from its drainage area (668 thousand km^2) as much as 1.6 billion tonnes of earth, the Ganges (drainage area of 1.1 million km^2) 1.46 billion tonnes, and the Mississippi displaces 0.3 billion tonnes of the most fertile soil every year. Large amounts of soil particles are transported by wind from dry land (as much as 200 – 800 million tonnes annually from Asia and Africa) to the oceans where these particles form sediments, as is documented by measurements taken at stations located on the islands in the Atlantic Ocean and by photos taken from satellites (BROWN et al., 1984).

Other sources of data are the areas of formerly cultivated land abandoned because of degradation by erosion and also records on the continuous decline of yields. Aerial photos provide evidence that deserts have expanded as a result of wind erosion: 650 thousand km^2 of land has become desertified over the past 50 years – HADAČ, 1987).

Generalisation of the data from direct measurement of the loss of soil through erosion in some countries, particularly the erosion data from India, China, the former USSR and the USA, where 52% of all the world's cultivated land is concentrated and where more than half of the food consumed in the world is produced, allows the excess loss of topsoil on the Earth (Table 1) to be estimated at above 23 billion tonnes annually (BROWN et al., 1984). Excess loss of topsoil due to erosion is defined as the amount of soil lost minus the amount of soil newly produced (on a global average, the threshold of excess loss is 12.35 tonnes per ha annually).

One-third to a half of the cultivated land of the world is cultivated without due care; owing to this, such soil has become a non-renewable resource. If erosion develops at its current rate (i.e., 0.7% annually), the carelessly cultivated areas mentioned will be totally deprived of their topsoil in a period of about 150 years. The data in Table 2 show that the population growth reduces not only the area of land per capita of population but also the volume or weight of the topsoil. If the world's population increases by 40%, as expected, the area of land per capita will shrink by 25% and the weight of the topsoil will be reduced by as much as 38%.

When the topsoil layer declines to a base level, the field is abandoned owing to poor crops or for economic reasons; this increases the exploitation pressure exerted on the remaining cultivated areas whose fate may then be the same. What farmers usually do when the topsoil layer diminishes is to continue tilling such land, adding the subsoil to what has remained of the diminishing topsoil; the

subsoil will successively prevail. Commercial fertilisers may partly replenish the nutrients removed by erosion but it is impossible to replace the lost organic matter and the lost part of the organic (and other biotic) life. All this results in a damaged soil structure, disturbed thermal and air regimes and reduced soil fertility.

Table 1

Estimates of excess losses of soil due to erosion in the world (BROWN et al., 1984)

Country	Total area of cultivated land	Excess loss of soil due to erosion
	10^6 ha	10^6 t annually
India	138	4 700
China	98	3 300
Countries of former USSR	248	2 300
USA	165	1 500
Total	649	11 800
Other countries of the world	599	10 900
Total	1248	22 700

Table 2

World soil resources and excess loss of soil (BROWN et al., 1984)

Year	Population	Cultivated land	Excess loss of soil	Remaining soil	Topsoil per person
	10^9	10^9 ha	10^9 tonnes annualy	10^9 t	t
1980	4.42	1.26	22.6	3500	792
1985	4.83	1.28	23.1	3385	701
1990	5.28	1.30	23.5	3270	619
1995	5.73	1.32	23.9	3150	550
2000	6.20	1.34	24.2	3030	489

The effect of erosion on the yields of cultivated crops can hardly be explained in simple terms because they are, as a rule, progressive and cumulative. A strong influence is exerted by the soil type and the depth of the soil profile. Hence, the annual loss of topsoil is almost negligible in soils 2 metres deep but in the case of soils where the topsoil layer is between 15 to 20 cm thick the conditions are entirely different and the threat is much greater – and the ill luck is that 15 to 20 cm is the most common topsoil thickness. Trials with an artificial removal of the topsoil were conducted in different places in the USA and the results showed that the yield was reduced by 53 to 80% in maize and by 68% in cotton (compared with control plots). During other trials it was found necessary to increase NPK fertilisation of

heavily eroded soils by 101 percent (from 20.1 kg per ha to 42.5 kg per ha – BROWN et al., 1984).

Erosion usually worsens the texture of the soil. Eroded soils usually have a higher compactness and are difficult to plough and cultivate (38% more fuel is required, on an average, for these operations). More fertilisers and more fuel mean higher costs; hence, other conditions of crop cultivation being the same, the actual price of the foodstuffs produced on eroded fields must be much higher than the food produced in fields with a lower erosion damage.

The problem of the reduction of yields in fields affected by erosion is closely associated with the issue of providing sufficient food and averting hunger on the Earth. Looking back through history, agriculture has often failed to adequately fulfil its mission: though of course the crops achieved depend primarily on the natural conditions, there is a number of other factors that can considerably influence the results of farming – these include the agricultural system used, the organisation of the land fund, the long-term erosion control measures taken and the over-all societal effects such as the general state of the economy, the demographic trends in the population, wars, etc.

Since World War II, the yields of farm crops worldwide have more or less doubled thanks to the use of new techologies. This has led to a general increase in food production; thus, the growth of crop production (about 3.1%) has exceeded the growth of the population (1.9%) and has, in escence, largely eliminated the periods of famine which repeatedly affected mankind in the past (for some notable exceptions, see following paragraph). Successful crop farming has been supported by comparatively favourable climatic conditions and (until 1973) by a copious supply of inexpensive energy.

It is hard to express in accurate figures the extent to which the population's need of food is met in different countries; it is also difficult to estimate the extent of malnutrition. The most obvious and acute manifestation of a lack of food is the people's dying of hunger, hence, these mortality figures may be indicative of the changes in the performance of agriculture as a result of the changes mentioned in the preceding paragraph: except in China where 8.98 million people died in 1960–1961, no famine occurred in the world between 1950 and 1970. Later on, however, 1 million people died in Nigeria (Biafra) in 1968–1969, 430 thousand people in Bangladesh in 1971–1972, 830 thousand in India in 1972, 100 thousand people in the Subsaharean (Sahel) area in 1973, 200 thousand in Ethiopia in 1972–1974, 330 thousand in Bangladesh in 1974, 450 thousand in Cambodia in 1979, 30 thousand in Ethiopia in1983, 3.5 million in Brazil in 1980–1985 (BROWN et al., 1984; HADAČ, 1987). Owing to increasing population and declining soil fertility, largely ascribable to soil erosion, the danger of famine has been faced (since 1984) by as many as 22 countries of Africa, Latin America and Asia with a total population of about 145 million.

The position and objectives of erosion control are now inseparably tied in with the mission of agriculture, hence also with the economic, environmental, demographic and socio-political conditions.

Soil erosion causes double harm: first, it reduces the upper fertile layer of agricultural and forest soils and second, it deposits unnecessary material along the bottoms of water courses and silts up reservoirs and river lakes. To remain

navigable, many rivers require costly dredging or building of walls. Removal of deposits is necessary on almost all water courses to avert the threat of flood. Protection of larger water courses and reservoirs against erosion includes the damming of the torrents in the upper part of the catchment basin. It sometimes happens, mainly in the developing countries, that serious financial losses are suffered when the designers fail to foresee the rapid silting up of reservoirs built as a source of irrigation water and often also to generate power.

1.2 Conservation of farm and forest land

The current need to conserve the land fund ensues from man's historical effort to exploit natural resources first for the sake of mere survival and later mostly to augment benefits and affluence. The first growers occurred in the Neolithic, 10 to 4 thousand years ago when – in the post-glacial climate – the average annual temperatures of the air on the Earth were 1 to 2 °C higher than today and successively decreased to the current level; the growers settled in fertile lowlands where they burned forests to change them into fields and grazed their livestock in grassy oak groves. The burning and clearing system and fallowing, practised by the early farmers in combination with forest grazing and hunting, gave rise to the agricultural and forest land fund. As the population grew, fields spread to the colder areas and hilly terrains; however, these farming activities devastated forests and altered their species structure because the thin oak groves were more suitable for grazing than the shady fir-and-beech forests and, in addition, oaks and pines were not so easy to burn.

There had been a transient decline in average temperatures before the Christian era began but later, during the 9th to 11th centuries, annual temperatures increased again to a level 1 to 1.5 °C above the annual average today: this allowed of planting vineyards 200 metres higher above sea level than today and of building settlements (having burnt the forest) at higher altitudes. Soils covered by forest or grass were safe from erosion but the small fields and mainly the deforested hillsides were badly affected, as is demonstrated by archaeological finds under the deep deposits of material transported by erosion processes from higher-altitude locations. Nevertheless, the population density was still low, so new land could be cultivated and wild vegetation could be left to overgrow the abandoned fields and restore their fertility.

The warmer time between the two cold periods of 1200–1250 and 1430–1450 saw an extensive colonisation of the central European territory as part of the general colonisation wave from the western parts of the continent. In the particular case of the Czech Kingdom, migrants were attracted by the economic situation in the country: using the feudal principle by which unsettled land belonged to the Sovereign, the Czech King donated free land to the nobility and the Church who, to draw maximum benefit from their new landed property, brought new settlers first from the overpopulated regions of the Czech lands and later from other countries; this in-migration trend lasted until the 14th century. The upsurge of economic activity (building new villages and towns, development of mining and

metallurgy, production of charcoal both for the artisans and as fuel for heating) required much wood and the area covered by forests was shrinking.

Farmers at that time practiced the three-field system, including a winter crop, summer crop and fallow. Hence, though the yields were the same as in antiquity, productivity was trebled. Pastures and forests were common. Part of the land was kept by peasants who had to pay the feudal rent and another part was kept by the feudal landlords (often free of any duties). Farming on small fields where cereals and legumes were grown (root crops were introduced much later, late in the 18th century) more or less prevented erosion, which thus endangered only the fields on slopes. In some forests, on the other hand, the situation was less favourable.

Forests were kept by several types of owners. The largest forests were owned by the feudal lords who used them only for hunting (no grazing and no felling were permitted in such forests). Some forests, however, belonged to the owners of mines and smelteries and to municipalities; such forests were commercially exploited.

The poor state of some forests in France, Switzerland, Italy, Bulgaria and in all countries of the former Austro-Hungarian Empire was conducive to disastrous floods which damaged farms, roads, fields and pastures in which they cut deep gullies. Larger water courses began to be regulated by reinforcing the banks and by fencing; flood control was usually combined with other purposes, including the use of water power and the possibility of log floating (making rivers navigable, building weirs, diversion channels and the like). This, together with the drainage of lowland wetlands, allowed new land to be cultivated. However, a substantial change could only occur when the real causes of floods and erosion – deforestation of the spring areas – were understood and explained. G. L. DE BUFFON was the first to offer such an explanation (in 1739). The care of forests was enhanced when government authorities began to be involved (the Theresian Forest Order, the State Forest Inspection Authority). Forestry schools were established, forestry literature began to be published. Increasing use of coal instead of wood (since the 18th century) was among the important factors which contributed to more consistent care of forests.

After the end of the era of feudalism, a new system of farming (crop rotation, following the pattern of the Norfolk rotation system) was introduced and root crops and technial crops began to be grown. The new crop-rotation system was more productive because the fallow could be excluded. The livestock which could be kept indoors produced organic manure to maintain the fertility of the soil. Soil fertility was also improved by deeper tillage because better ploughs and other improved implements became available. Mass use of forest litter was among the adverse consequences of the year-round housing of the livestock. The agroindustries began to develop rapidly (sugar mills, breweries, flour mills and others).

Agricultural intensification increased the danger of erosion (ZACHAR, 1970). Ravines and gullies were cut by flood rains in some fields, wind and water erosion thinned down the topsoil layer, floods spread silt on the surface of cultivated fields. To control these processes, the regulating authorities passed laws on re-forestation and on torrent control. In the former Austro-Hungary, Law No. 116 on the Soil Improvement Fund and Law No. 117 on safe diversion of mountain waters were passed in 1884. On the basis of these regulations, torrents and gullies could be

controlled and larger-scale erosion control measures could be taken in the catchment areas of these water courses.

Mankind's vital dependence on the soil often led to the development of sentimental ties to the land: the owners wanted to maintain or improve the fertility of their land for their descendants. These efforts included drainage of wetlands, unification of smaller plots into large fields, the building of smaller-scale irrigation schemes, etc.; all this, as a rule, involved erosion control.

CHAPTER 2

Erosion of the Soil

The term erosion has its origin in Latin and its original concept included only the initial stage of what is today considered as the erosion process. The concept of erosion now comprises the whole process which is induced by exogenous factors on the soil surface and consists in the destruction of the earth's (soil) surface by the kinetic energy of exogenous factors (e.g., water or wind), release of separate particles and transport of this material to another place (usually below the original site) where such material, or at least part of it, is deposited. The intensity of accumulation of eroded material depends on the extent of the decline in the energy of the factors by which the erosion is caused.

Depending on the exogenous causative factors, erosion processes have been taking place incessantly on the soil surface. The processes which contributed to the formation of the soil surface during the different geological periods are called historical or geological erosion. The processes currently taking place are called contemporary erosion and are the subject of erosion control.

2.1 The types of erosion

The types of erosion are classified according to the factors by which the erosion processes are caused. Water and wind are the most common and most important causative factors and accordingly, water erosion and wind erosion are considered as the two basic erosion types.

However, the term water erosion only applies to erosion processes caused by water in its liquid form; water in its non-liquid forms may cause glacial erosion and snow erosion.

There are still another two separate types of erosion, namely soligenous erosion and athropogenous erosion (Fig. 1). The former is caused by the movement of a mixture of scree and water in which the action of the scree prevails, thus justifying the classification of soligenous erosion as a special erosion type. In the case of anthropogenous erosion the causative factor is man with his contribution to the intensification of the two key factors, water and wind. The definition of the type of erosion depends on the identification of the causative factors which often combine and interact.

2.1.1 Water erosion

Water erosion includes the destruction of the soil, loosening of the soil particles and their removal with the runoff, washing out and flushing of the soil with water running on the surface by which it is then deposited and accumulated in another

Fig.1 A grass stand on a sloping pasture damaged by grazing cattle.

Fig.2 A slope damaged by sheet and scouring erosion (Photo F. Toman).

place (Fig. 2). This type of erosion occurs in areas where flood rains or abrupt thaws cause a rapid surface runoff.

When rain drops hit the soil surface, their kinetic energy breaks the soil crumbs and aggregates. If there is more rainwater than the soil is able to absorb, the excess water is accumulated on the soil surface. The disintegration of the soil crumbs and aggregates into smaller crumbs and even into separate soil particles also takes place when snow thaws. These processes are especially distinct in soils with an unstable crumbly texture. The loosened small crumbs and soil particles are transported by the water running on the surface down the slope to the lower locations where the slope is milder so that the transported material can be intercepted and almost completely or at least partly deposited (Fig. 3). What is not deposited is carried on into the streams and water courses. During the first stage of the erosion process, nutrients are washed away together with the topsoil, thus further reducing the fertility of the soil on hillsides.

When it starts raining there is no surface runoff because all the rainwater is absorbed by the soil. As a rule, the soil has the highest absorbing capacity at the onset of the rain but this is progressively lost as the rain continues. As soon as this capacity declines to the level of the intensity of the rain, the soil fails to absorb all the water, so that some water is first accumulated on the surface and later it runs off down the slope (if the terrain is not flat).

When snow thaws, the soil usually remains frozen and only the top layer yields to thawing. In soils with a less-developed crumbly texture the absorbing capacity is very low, more or less negligible, under such conditions, so the surface layer is

Fig.3 A maize stand damaged by deposited earth material (Photo J. Filip).

soon oversaturated and allows the rest of the snow water to run away, often carrying away a large quantity of soil particles.

Water erosion increases down the slope from the watershed towards the bottom of the valley. There is usually no erosion on the watershed divide because there is not enough accumulated water to transport the earth; however, down the slope the amount of water running on the surface, its speed and tangential stress increase and so also does the intensity of erosion.

Erosion processes (usually their more complex forms) may also be caused by a concentrated flow of water from urban areas and communication networks or by other factors associated with human activity.

2.1.2 Wind erosion

Wind erosion is usually caused by the kinetic energy of the wind; it consists in the destruction of the soil or rocks and transportation of the loosened particles from their original location to another site (Fig. 4) where the material is deposited. This harmful activity of the wind is very dangerous mainly in arid regions and in easily drying soils having a powdery structure. Owing to its dependence on the aridity of the affected regions, wind erosion is less dangerous than water erosion in cultivated areas.

Strong wind cuts the soil, fine sand and nutrients from surfaces which are either bare or just sparsely overgrown with vegetation and transports them sometimes to very distant areas. At lower windspeeds the soil is often deposited on adjacent

Fig.4 Wind erosion on a ploughed field (Photo M. Pasák).

surfaces in gullies, trenches, etc. The losses of soil owing to wind erosion are particularly heavy in the steppe regions originally ovegrown with grass but now used as arable land mostly for continuous cropping.

The effects of wind on the soil surface are similar to those of water. The kinetic energy of the air masses loosens soil particles and sets them in motion. The mode of the transport of the soil particles mainly depends on the speed of the vertical and horizontal movement of the air close to the soil surface and on the size of the soil particles.

The largest soil particles are pushed or rolled on the soil surface; smaller particles move by jumping (saltation) and the smallest ones are carried on by the air masses.

2.1.3 Glacial erosion

Glacial erosion is caused by the gravity forces which develop as a glacier moves down a hillside along the base rock; it also comprises the transport of the rock material by the glacier and its deposition in the form of moraines.

The base rock is abraded and polished by both the glacier and the boulders it carries. Depending on the place of deposition, the moraines may be either upper, lying on the surface of the glacier, lower, lying on the surface of the ground under the glacier, or lateral on the glacier's sides. The material (debris) deposited in places where two glaciers are in contact forms central moraines and that on the foothills forms the front moraines.

The majority of the debris carried by the glacier remains in the places where the moraines developed and only a small proportion of the material gets into the brooks and rivers.

Under European conditions, glacial erosion is taking place in the high mountains.

2.1.4 Snow erosion

Snow erosion is caused by gravity forces when masses of snow are set in motion, most frequently in the form of avalanches. Avalanches loosen the rock and earth material and the vegetation, carry this debris away and deposit it at the foot of the moving mass of snow. The occurrence of snow erosion is limited to mountain regions.

2.1.5 Ground erosion

Ground erosion is caused by debris oversaturated with water and consists in the destruction of rock and earth material on the bottom of the debris streams. Like snow erosion, the occurrence of ground erosion is also largely confined to mountain regions.

2.2 Manifestations of erosion

Manifestations of erosion are defined as separate forms and formations whose size and extent depend on the intensity of action of the exogenous causative factor. With water erosion, these include, first of all, formations at the place from where the material is being removed and mostly those on the surface of the earth. Subsurface erosion is exceptional and the possibilities of its occurrence will be dealt with separately. On the other hand, wind erosion is mainly a matter of formations occurring in the places of deposition and accumulation.

2.2.1 Manifestations of water erosion

Manifestations of water erosion are classified according to the rate of surface runoff. First there is areal erosion, comprising the simplest forms of erosion. Then, as surface runoff concentrates on the slope, scouring erosion occurs. The processes by which the beds of water courses are eroded are referred to as stream erosion. Hence, the three basic forms of water erosion are: areal erosion, scouring erosion and stream erosion.

Areal erosion comprises the simplest erosion forms which as a rule occur on a larger area of the soil surface. Areal erosion starts close to the watershed divide and may occur in any of the following forms: selective, sheet, rill and furrow erosion.

Scouring erosion either starts at a certain distance from the watershed, following up with furrow erosion, or may, for example, result from the concentration of a larger amount of precipitation water running down the surface of unpaved roads ets. The area of its occurrence reaches up to the banks of the water courses. The key trait from which the name of this type of erosion is derived is the scouring action of running water. Scouring erosion can be subdivided into gully, pothole and ravine erosion.

Stream erosion is confined to the beds of water courses. It can be subdivided according to the place of occurrence within the stream bed (bank erosion, bottom erosion) or according to the nature of the stream (torrent erosion, river erosion).

The term bank erosion also applies to the destruction of the banks of larger water bodies where it is caused by wind-driven waves.

2.2.1.1 Areal water erosion

On the upper part of a regular slope, water running down the slope is distributed more or less regularly on the surface, causing what is called sheet erosion. However, the water soon concentrates in a number of runoff lines. The runoff speed is still comparatively low so that the loss of soil carried away by the water is small; the first soil constituents to be lost are the finest particles and, especially, the plant nutrients. This simplest form, called selective erosion, is very dangerous because it changes the texture of the soil by removing the fine soil particles and deprives the soil of plant nutrients. Most of these effects cannot be observed by the naked eye. In selective erosion, particles may be shifted to very short distances to

Fig.5 A field affected by furrow erosion (Photo M. Uličný).

fill microdepressions. Such sediments are always deprived of the above-mentioned constituents.

When there is a larger amount of rainwater running down the surface of a slope (especially after flood rains on arable land loosened to a small depth, e.g., by shallow cultivation or tillage), excess moisture accumulates in the upper horizon if the absorbing capacity of the underlying soil horizons (or the subsoil) is very low. A soaked topsoil horizon is easily carried away by water. A similar situation occurs when the water from thawing snow runs down, carrying with it the slushy thawed upper layer of topsoil. This phenomenon is called sheet (layer) erosion (Figs. 5 and 6), because the soil moves in a complete layer.

Rill erosion and furrow erosion are somewhat more complex forms of areal water erosion. They are classified within areal erosion because of their areal occurrence though some authors include them among the manifestations of scouring erosion (erosion rills and furrows do in fact arise as a result of the scouring activity of water). Rill erosion starts at some distance from the watershed or from the upper end of the field in which surface runoff occurs: areal runoff successively divides into lines which then join one another to concentrate the runoff; this leads to a higher flow rate and higher tangential stress which in turn are conducive to a higher erosion intensity. Small rills are scoured in the soil and furrows develop below their confluence. These manifestations of water erosion badly affect arable

Fig.6 Transition of furrow erosion into sheet erosion (Photo M. Uličný).

Fig.7 Transition of an erosion rill into a gully (Photo J. Polák).

fields because they significantly reduce the thickness of the topsoil layer, thus diminishing the fertility of such soils.

Areal water erosion causes considerable losses in agricultural production. Tillage and other cultural practices level out the rills and furrows, thus removing the evidence of erosion. The thinning of the topsoil layer remains unnoticed because the material removed by erosion is replaced by admixing subsoil with the topsoil during tillage. The natural fertility of the ploughed layer declines progressively.

Part of the eroded material is deposited in the lower parts of the affected fields where it often damages or destroys the crops and, having an unfavourable composition, also causes a reduction of soil fertility.

2.2.1.2 Scouring erosion

Scouring erosion is caused by the concentration of a larger amount of water running down the slope. Gullies, potholes and ravines develop progressively in the sloping fields exposed to scouring erosion. These erosion features cannot remain unnoticed and cannot be levelled out by year-to-year cultural practices. The lowest degree of scouring erosion is gully erosion which as a rule occurs during a single flood rain. The gullies make it difficult to cultivate the land: a depression develops progressively in such a place and then cultivation has to be given up. Thus, the affected slope is divided into smaller fields and the area of arable land shrinks; costly reclamation efforts are required if such gullies are to be removed. The shape of the gully depends on the nature of the soil profile. Its walls are usually

Fig.8 Reclamation work started along a gully (Photo J. Antal).

Fig.9 Deposited material from a field affected by rill erosion (Photo M. Uličný).

Fig.10 Scouring erosion caused by the concentration of surface runoff along the boundary of a field (Photo J. Sanetrník).

vertical and in loose earths (loess) its depth may be as great as several metres. Water often cuts its way underground in such places: this erosion feature is called tunnel erosion.

Pothole erosion is usually referred to as a higher stage of scouring erosion; however, the difference between the pothole and the gully is not clearly defined, so this type of erosion is sometimes left out in systematic classifications.

Ravine erosion is considered as the top stage of scouring erosion: the ravine remains active for a long time and its cross-section is usually V-shaped.

It is also characteristic of scouring erosion that the formations grow in both depth and length (up the slope). This process is usually referred to as retrograde erosion.

Part of the material removed from the gullies and ravines is deposited at the foot of the slope where it forms dejection cones of coarse-grained material. The remaining material is carried away to the water courses.

The development of any form of scouring erosion depends on the structure of the soil profile (Figs. 10 and 11) and on the underlying rock. The important factors of the rocks include the degree of weathering, the mode of separation of weathered parts and the strength of the rock itself.

2.2.1.3 Stream erosion

Stream erosion is caused by the action of the water stream in torrents or rivers. Depending on the nature of the water course, there is either torrent erosion or river erosion and depending on the focus of the action of stream erosion there is either bottom erosion (destruction and/or deepening of the water course) or bank erosion (destruction of the water course's banks mainly by the water stream). Both the bottom and the banks are eroded in the torrents in which the occurrence of bank erosion usually depends on the erosion of the bottom. With river erosion, the erosion of the banks prevails, taking the form of bank cavings.

Fig.11 Erosion rills along the thalweg of an arable area (Photo J. Polák).

The consequences of stream erosion include the transport of material removed from soil surfaces into water courses and the loosening and transport of material from water courses' beds and banks.

These forms of erosion are confined to the beds of water courses on the surface of the Earth. Submersions underground are an exception: they mostly involve the effects of the mechanical and chemical activity of water. Such effects are usually classified as geological erosion but sometimes they are placed under the heading of subsurface erosion which also comprises vertical erosion (shifting fine soil particles, especially nutrients, to the lower soil horizons) and tunnel erosion.

2.2.2 Manifestations of wind erosion

The manifestations of the action of wind on the soil surface include deflation, i.e., the loosening of earth particles (abrasion) and their transport, and corrasion, i.e., the scraping of rock surface by rock fragments transported over it by wind.

Wind erosion is less important under European conditions; however, when it occurs it always affects a large area.

2.2.2.1 Deflation

The removal of loose earth particles by wind, referred to as deflation, is subdivided according to the mode by which the particles are driven or carried by wind: thus we recognise shifting erosion and dust storms.

Shifting erosion means the shifting of soil and earth particles to short distances on the soil surface: driven by wind, the particles may glide or roll or jump (saltate) on the soil surface.

Dust storms may vary with the particle size of the blown material (dust or sand storms) and with its colour (black storms), etc.; during such storms, air masses may carry the material at a great height and over long distances (even several thousands of kilometres) from the erosion focus.

Shifting erosion usually occurs is sandy areas where the soil surface is not protected by vegetation. The material is shifted in the wind's direction and is deposited in the form of ripples, dunes, crescentic dunes, etc. These formations usually occur in maritime areas, in deserts and in the valleys of large rivers.

Dust storms occur in large arid regions, mainly in deserts and semideserts from which, far back in geological history, material was blown to the present-day loess regions.

2.2.2.2 Corrasion

Corrasion depends on the source of particles carried by wind and on the hardness of the rocks exposed to the corrasion processes. The most conspicuous and most common formations produced by this type of erosion are sandstone columns of different shapes and sizes in sandstone areas.

2.3 Erosion Intensity

Exogenous forces continuously change the relief of the Earth. This process is slow if it takes place under natural conditions; as such it is called natural or normal erosion. When measured by the human life span, it is unnoticeable, hence almost harmless.

However, if the natural processes are disturbed (man's activities are most frequently the cause of such disturbances) the erosion may intensify and become dangerous. Such erosion processes are referred to as accelerated erosion.

The intensity of the erosion is as a rule measured by the thickness of the surface soil layer lost or by the amount of soil removed or washed away from a unit area per unit of time. The amount of removed material is expressed either as weight or volume. In the case of scouring erosion, erosion intensity is measured in the same way as in the hydrographic network, i.e., by the density of gullies, potholes and ravines (the total length of these formations per unit area).

2.3.1 Normal erosion

The main feature of normal erosion is its low intensity, ensuing from the slow action of the erosion factors. Normal erosion is almost exlusively caused by primary factors: it does not afford conditions for any significant involvements of the human factor.

The loss of soil should remain balanced with the formation of new soil matter: the soil loss should not exceed soil increment. Though some quantitative changes may occur in the upper soil stratum, its thickness should not be affected.

Normal erosion can take place mainly in soils with a permanent vegetation cover; on arable land it involves only the lowest erosion manifestations. Seasonal erosion is also often classified as normal erosion.

2.3.2 Accelerated erosion

Unlike in normal erosion, the human factor is strongly involved in accelerated erosion: through his activities, man intensifies the action of a number of erosion factors so that the loss of soil matter is faster than the formation of new soil. The adverse effects of the human factor can be encountered not only on farmed land but also on forest soil and in places affected by urbanisation. The erosion processes of the higher forms of scouring erosion contribute significantly to accelerated erosion.

2.3.3 Admissible intensity of erosion

Determination of the admissible intensity of erosion has mostly been used in the assessment of the bearability of areal erosion; hence, it is based on the values of the natural increment of soil. Data on the natural soil increment vary within a very wide range, usually between 0.025 and 0.125 mm per annum: almost all the data published by different authors (HOLÝ, 1978) remain within this range.

The natural soil increment provides the basic data for the determination of the admissible intensity of erosion. However, this criterion considerably reduces the possibilities of the use of soil for farming; hence, two other criteria are also used: the depth and fertility of the soil profile. The greater the depth, the higher the limit of admissible intensity of erosion.

In Central European conditions, the admissible loss of soil ranges from 1 to 10 tonnes per ha annually.

The intensity of scouring erosion depends on a number of local factors and the values in which it is expressed have only regional or local applicability; they are not used for preparing actual anti-erosion measures.

CHAPTER 3

Erosion Factors

3.1 Definition of erosion factors

Soil erosion is a result of the action of erosion agents (water, wind, ice and others) on the soil under actual natural conditions or conditions induced or modified by man.

The natural and anthropogenous conditions exerting the greatest influence on the rise, development and results of the erosion process are called erosion factors.

The most important erosion factors include the climatic, hydrological, topographic, soil, geological and vegetation conditions, as well as the economic and technical and the socioeconomic conditions of the human society.

None of the erosion factors ($X_1, \ldots X_n$) act in isolation: they always combine and interact to influence the mechanism of the erosion processes. Mathematically, this relationship can be described by the general equation of the erosion process:

$$S_p = f(X_1, X_2, \ldots X_n), \tag{3.1}$$

where S_p – intensity of soil erosion in the area under study ($t \cdot ha^{-1} \cdot year^{-1}$; $m^3 \cdot ha^{-1} \cdot year^{-1}$; $mm \cdot year^{-1}$),

$f(X_n)$ – effect of the nth erosion factors on soil erosion intensity under the given conditions of the area under study (for details see Chapter 4.3).

Though the erosion factors combine and interact, it may happen under actual conditions that the effect of one factor or a group of factors will prevail and become a factor (factors) of key importance.

The influence of man's activity on the world of Nature is generally increasing, including also an intensifying influence on the erosion processes. Man's activities influence not only the "man-made" group of erosion factors, such as the economic and technical conditions and socioeconomic conditions, but also some of the natural factors, e.g. vegetation and (to some extent) the topography of whole regions.

Under Central European conditions, where water and wind are the main erosion factors, it is necessary to study in detail the influence of the different erosion factors on the intensity of the water and wind erosion of the soil.

3.1.1 Climatic conditions of the territory

The key climatic characteristics influencing erosion processes in a given territory (either directly, i.e., through their direct action on the soil, or indirectly) include atmospheric precipitation, wind, air temperature, air humidity and solar radiation.

The factors having a direct effect on soil erosion are atmospheric precipitation (water erosion) and wind (wind erosion).

The remaining climatic characteristics of the territory, including air temperature, humidity and solar radiation — and also wind in the case of water erosion and precipitation in wind erosion — affect erosion indirectly, for example through their influence on the process of evapotranspiration, on the size of rain drops and the angle of their incidence on the soil surface, on soil moisture content, soil texture, on the rate of snow thawing, etc. Climatic conditions also indirectly influence soil erosion by their action on vegetation in the given territory: the better the conditions for plant life, the lower the effect of erosion factors on the soil will be. (Fig. 12.)

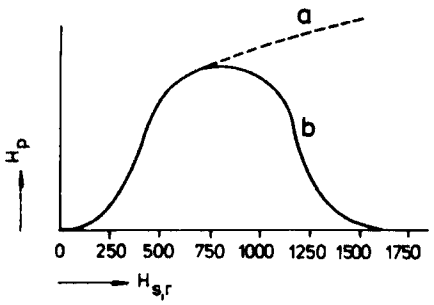


Fig.12 Qualitative effect of atmospheric precipitation on the intensity of water erosion of the soil: a — without vegetation, b — with vegetation (after HUDSON, 1971).

The vertical atmospheric precipitation originating from the condensing of water vapours in the air falls onto the ground either in liquid or solid state.

Division of precipitation by the place of origin and state of aggregation is shown in Table 3; it should be borne in mind in this context that horizontal precipitation is not taken into account under Central European conditions because it makes no significant contribution to surface runoff.

Table 3

Distribution of precipitation by place of origin and by state of aggregation (KLOPČEK and ANTAL, 1985)

Precipitation			
horizontal		vertical	
liquid	solid	liquid	solid
Mist		drizzling rain	snow
			freezing rain
Moistening	hoarfrost		granular snow
	ice accretion		crystalline snow
Dew	rime		crystalline ice
	glazed frost		hail
			ice needles

As found, the total annual precipitation level expressed, for example, by the annual precipitation sum ($H_{S,r}$ is not the key factor responsible for the intensity of erosion: the key factors are, in particular:

– the time distribution of rains heavy enough under the existing conditions to cause surface runoff of rainwater (See Sections 3.1.2 and 3.2.1),

– the development of the thawing of snow,

– the distribution of the annual value of the so-called rain factor R (See Section 3.2.2) over the year.

Flood rain, characterised, among other factors, by a high intensity and sporadic occurrence in periods when the vegetation cover is not yet strong enough to afford adequate protection of the soil surface, may cause disastrous erosion even in climatic regions whose total annual precipitation is typical of arid climates ($H_{S,r} < 400$ mm).

On the other hand, even humid climatic regions ($H_{S,r} > 600$ mm) may be safe from damaging water erosion of the soil if the rains such regions receive do not cause surface runoff or if heavy rains capable of inducing surface runoff occur when the soil surface is adequately protected by the vegetation cover.

A similar explanation can be used in describing the effect of the time distribution of the so-called erosion-active winds on the intensity of the wind erosion of the soil. The erosive action of wind is modified by the vegetation cover of the soil and depends also on the moisture content and homogeneity of the soil surface; hence, wind erosion may be harmful even in areas with a small number of windy days if the erosion-active winds occur in periods during which the soil surface is dry, broken (e.g., by frost, tillage, etc.) and bare (not protected by a vegetation cover).

In Central Europe (the temperate zone) the soils are most vulnerable to the erosive action of water and wind during the spring season when the plant cover is still poor or absent, and the soil surface is disturbed by winter frosts, by cultivation, by fast thawing of snow, etc. However, erosion may also occur in winter, caused mainly by black frosts, and in summer as a result of frequent flood rains.

On the other hand, the danger of water and wind erosion in the temperate zone is low in autumn, as is confirmed by the data on the occurrence of silt in rivers in different seasons of the year.

3.1.2 Hydrological conditions in the territory

The hydrological conditions of the territory influence mainly the water erosion of the soil. Different forms of water erosion arise at different intensities (surface or subsurface erosion, areal or scouring erosion, damaging or harmless erosion, etc. – see Section 2), depending on the amount of water, on the speed and duration of its movement and on the environment in which the water moves.

Surface erosion of agricultural land is the primary issue from the point of view of soil protection; hence, the greatest importance is attached to the local hydrological characteristics that influence the surface runoff of precipitation water.

Surface runoff occurs when the intensity of rain or of thawing overrides the infiltration capacity of the soil, i.e., if it holds that:

$$i_s > v_i \tag{3.2}$$

where i_s – rain or thawing intensity ($mm \cdot min^{-1}$)
v_i – intensity of water infiltration into the soil ($mm \cdot min^{-1}$)

Surface runoff depends on two groups of key factors which exert their influence on surface runoff characteristics such as, for example, the rate of flow of the water running on the surface – v_p, the specific surface runoff – q_p, the length of time during which surface runoff continues – t_c and others. The first group of key factors includes the *characteristics of atmospheric precipitation*, especially the length of the duration of rain – t_d, rain intensity – i_s and the areal distribution of rain (precipitation).

The other group of key factors on which surface runoff depends includes the *characteristics of the drainage basin* (territory), particularly the size and shape of the drainage basin (territory), its soil conditions and plant life, its topography and the human activities practiced there.

The effects of the different factors on surface runoff characteristics, together with the quantification of the key surface runoff characteristics, are analysed in section 3.2.1.

3.1.3 The topography of the territory

The topographic characteristics having the greatest influence on soil erosion include the gradient, length, shape and exposure of the slope.

The average slope gradient can be expressed by any of the following equations (Fig. 13):

$$I = \frac{\Delta H}{\Delta L} \qquad (m \cdot m^{-1}) \qquad (3.3)$$

$$I = \frac{\Delta H}{\Delta L} \cdot 100 \qquad (\%) \qquad (3.4)$$

$$I = \frac{\Delta H}{\Delta L} \cdot 1000 \qquad (‰) \qquad (3.5)$$

$$I = \text{invtg } \alpha = \text{arctg } \alpha \qquad (°) \qquad (3.6)$$

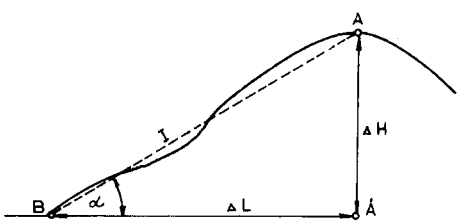


Fig.13 Diagram for the calculation of the slope gradient of an area.

where ΔH – the height difference between the end points of the slope under study (m),

ΔL – horizontal distance between the end points of the slope under study (m),

α – the angle between the plane of the slope and the horizontal plane (°)

The gradient of slope expressed as percentage – $I(\%)$ – can be converted to that expressed as degrees – $I(°)$ – (and vice versa) by the following equations:

$$I(\%) = 100 \text{ tg } [I(°)], \tag{3.7}$$

$$\text{or } I(°) = \text{arctg } [I(\%)\, 100]. \tag{3.8}$$

Relationships (3.7) and (3.8), i.e., conversion between $I(\%)$ and $I(°)$, are shown graphically in Fig. 14.

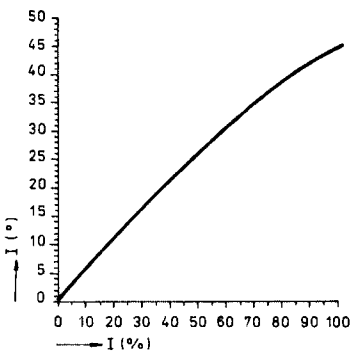


Fig.14 Graphical representation of equation (3.7) or also (3.8).

The length of a slope is defined for erosion-control purposes as the length of uninterrupted flow of water on the soil surface or as the length of a horizontal projection of slope lines between two natural or artificial obstacles to the flow of water down the slope under study (Fig. 15).

Slopes may be straight, convex, concave or combined (Fig. 16), depending on the shape of their cross-section. Each of the shapes influences the process of water erosion in its specific way, though all may have the same average gradient (Section 3.2.4.1).

Depending on the shape and gradient relationships on both the longitudinal section and cross-section, the surface runoff either concentrates or disperses on the slope (Fig. 17).

An indirect influence on the processes of both wind and water erosion of the soil is also exerted by the exposure of the slope: the light, temperature, moisture and wind-exposure conditions on the slope may vary with the slope's position in relation to the cardinal points.

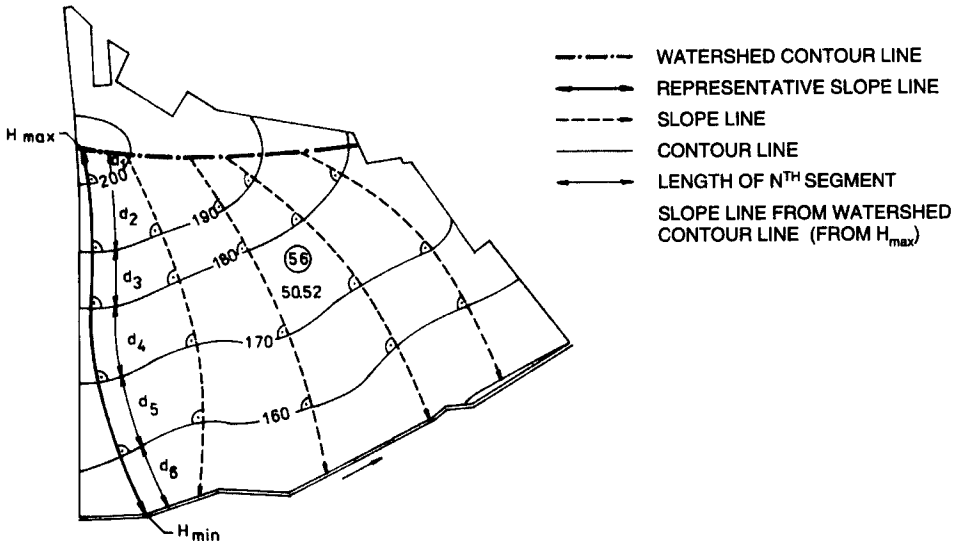


Fig.15 Diagram of the uninterrupted length of water flow along a soil surface.

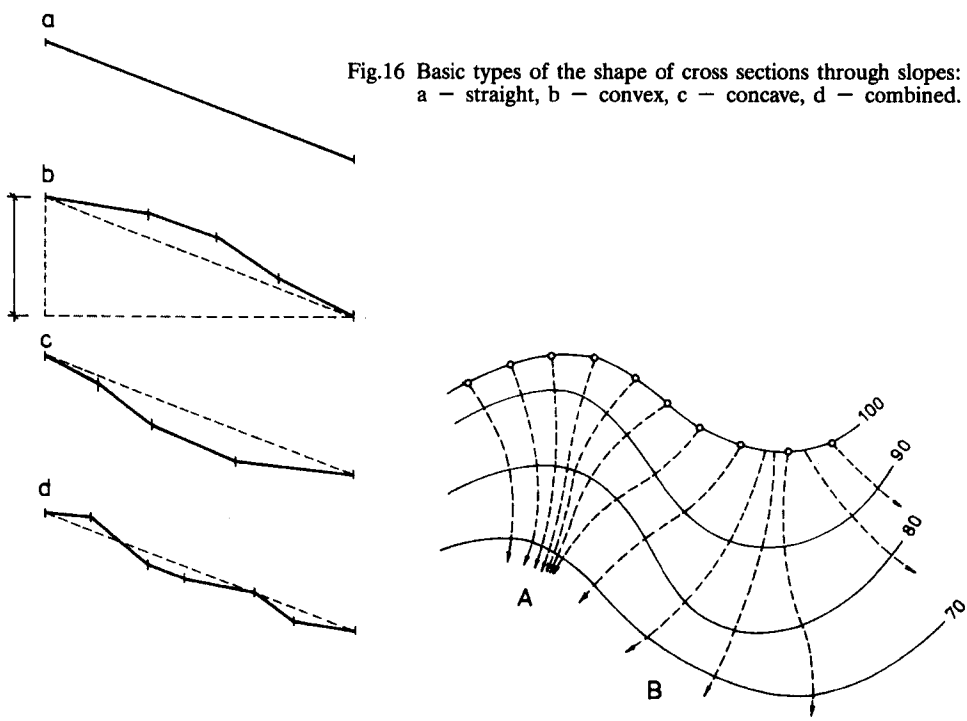


Fig.16 Basic types of the shape of cross sections through slopes: a – straight, b – convex, c – concave, d – combined.

Fig.17 Effect of slope shape and gradient on the movement of surface water.

3.1.4 Soil conditions in the territory

For anti-erosion purposes, it is necessary to know the soil characteristics that control:

- the infiltration capacity of the soil,
- the resistance of the soil to the destructive action of erosion agents,
- the resistance of the soil to the transporting action of erosion agents,
- the admissible intensity of soil erosion.

As found from theoretical analyses and experimental studies, these characteristics are as follows:

- soil grading,
- soil texture,
- soil permeability,
- humus content,
- moisture content and / or moisture regime of the soil,
- depth of soil profile.

General data on the above-mentioned characteristics can be read from the maps of soil varieties, soil types and the classified soil-and-ecological units.

3.1.5 Geological conditions in the territory

The soil characteristics are very closely associated with the geological conditions of the territory. Hence, they are often studied together — as the soil and geological conditions of a territory.

This is so because the properties of soils (grading, texture, content of minerals, depth and others) directly depend on the nature of the parent rock.

This effect of geological conditions on soil properties, combined with their effect on the hydrological conditions, makes up the indirect action of geological conditions on the erosion processes.

A direct action of geological conditions on erosion processes manifests itself mainly in places where the rock is exposed to a direct action of erosion agents, e.g., where the soil lacks (for any reason) a soil cover which normally protects it against the action of erosion agents.

3.1.6 Vegetation in the territory

For anti-erosion purposes it is necessary to know the characteristics of the soil's vegetation cover that control:

- the interception capacity of the aerial parts of plants,
- the absorption of the kinetic energy of water and wind,
- the roughness of soil surface,
- soil texture, permeability and stability,
- the moisture regime of the soil,
- the characteristics of the surface runoff of precipitation water.

The above-mentioned functions of the vegetation cover depend, first of all, on:

- the density, height and weight of the aerial parts of plants,
- the density, depth and weight of the root system of plants,

- the time and position of the growing season within the year,
- the cultural practices used and the way in which the soil's vegetation cover is used (mainly in cultivated plants),
- the amount and quality of plant residues (mainly in wild plants).

An example of the development of the aerial and underground parts of different farm crops – and thereby also the development of their protective action on the soil – is shown schematically in Fig. 18.

As described below, the condition of plant life in the territory are among the factors through which man can most intensively influence the erosion processes, both positively and adversely.

3.1.7 The economic and technical conditions in the territory

The following economic and technical conditions have the strongest influence on erosion processes:

- the mode of soil utilisation and management,
- selection of the agricultural crops to be grown,
- the sequence of different crops grown (crop rotation),
- the location of the fields with each crop in the territory,

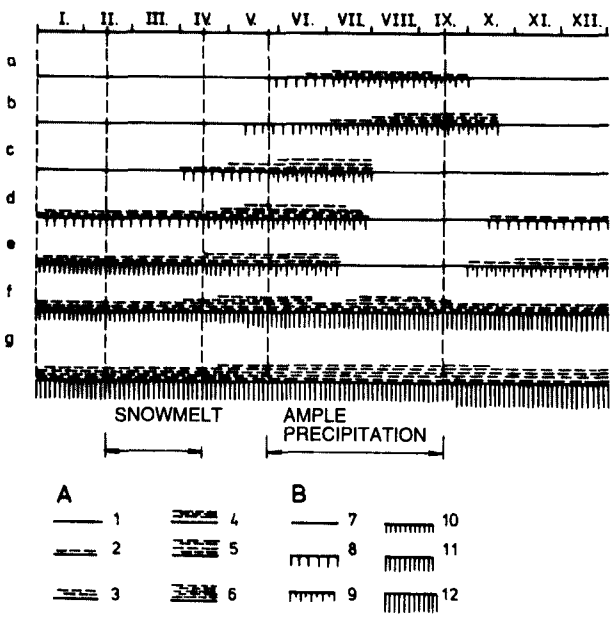


Fig.18 Development of the underground and aerial parts of the vegetational cover of the soil:
A – state of the vegetational cover: 1 – soils with no vegetational cover, 2 – very thin cover, 3 – thin, 4 – medium, 5 – good, 6 – very good,
B – fixation of the soil by roots: 7 – soil with no roots, 8 – very poor fixation, 9 – poor, 10 – medium, 11 – good, 12 – very good. (After RIEDL and ZACHAR, 1973.)

– cultural practices used in growing the crops,
– the structures built on the territory (roads, drainage systems, terraces etc.).

As follows from this survey, a change in the economic and technical conditions in the territory will influence the majority of erosion agents, particularly the hydrological, topographic and vegetation factors.

Special attention should be paid to the analysis of erosion processes when the natural plant cover of the soil is removed, e.g., when forest is changed into a field or when a permanent grassland is ploughed up, etc.; the same applies to all cases in which any human activity changes the runoff conditions in the territory, especially if such a change leads to a concentration of surface runoff.

3.1.8 The socio-economic conditions

Like the use and protection of any natural resources, the utilisation and conservation of soil also depends on the general development of the society and its social order.

Where the socio-economic conditions are favourable, the attitude to the soil is highly positive, allowing optimum management of the land fund. If any change occurs in the erosion factors it should not be left to cause increased damage: on the contrary, all damaging consequences of erosion should be minimised.

Favourable socio-economic conditions create prerequisities for the practical use of pertinent scientific knowledge for effective erosion control.

3.2 Evaluation of erosion factors

The main aspects evaluated in the different erosion factors or groups (combinations) of erosion factors include their effect on the runoff of precipitation water, on the erosive activity of water and wind, on the soil's ability to resist the erosive action of water and wind and on the intensity of the erosion process.

3.2.1 Runoff of precipitation water

Runoff is the part of precipitation water that gets into the hydrographic network (water courses, canals, ditches, etc.). Depending on the routes along which precipitation water gets into this network, total runoff (O_c) falls into three parts:

1. *surface runoff* (O_p), also called overland runoff: the part of total runoff leaving the territory by running over the surface of the land;
2. *subsurface runoff* (O_h), also called hypodermic runoff: the part of total runoff leaving the territory by running under the surface without contact with groundwater;
3. *groundwater runoff* (O_z), also called *base runoff:* the part of total runoff leaving the territory with groundwater.

Rainfall runoff, i.e., the surface runoff of rainwater, is the most important runoff category from the viewpoint of erosion control. Surface runoff caused by flood rain (rain which is usually short but heavy and confined to a small area) is the most dangerous kind.

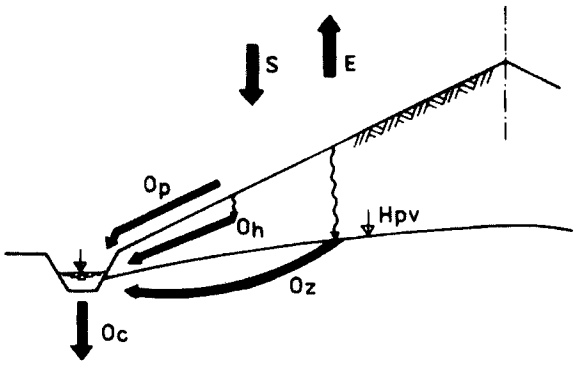


Fig.19 Diagram of the distribution of the total runoff of precipitation water.

Compared with the intensity of flood rain, the intensity of snowmelt is much lower; hence, the surface runoff from melting snow is lower than surface runoff from rain.

To be able to determine the characteristics of precipitation water runoff it is necessary to know, among other things, the characteristics of the so-called substitute design rain, the infiltration capacity of the soil, and the conditions of occurrence of surface runoff as functions of the intensity of design rain (i_S or q_S), of the duration of design rain (t_d) and of the infiltration capacity of the soil [$v_{i,t} = f(t)$, or $V_{i,t} = f(t)$].

3.2.1.1 Characteristics of design rain

For project-design purposes, characteristics of design (fictitious) rains were derived from a mathematico-statistical analysis of the characteristics of actual (natural) rains.

Design rain which is assumed to have the same effects on surface runoff as actual rain is defined by the following characteristics:

1. constant intensity (i_S) or abundance (q_S) during the whole duration of the rain,
2. time (duration) (t_d),
3. interval of occurrence (periodicity) (p).

The interdependence of the intensity, duration and periodicity is recorded graphically as the line of design rain intensities (Fig. 20).

To construct the lines of design rain intensities for an actual territory, any of the following three procedures can be used:

- ask the Weather Service to supply the pertinent simulated rain characteristics;
- use the rain-gauge records taken in the loality under study and interpret them by the Reinhold method (DUB et al., 1969),
- calculate the required data using the terms which express the courses of the lines of design rain intensities.

There are four formulae by which the lines of design rain intensities (abundances) are most frequently calculated:

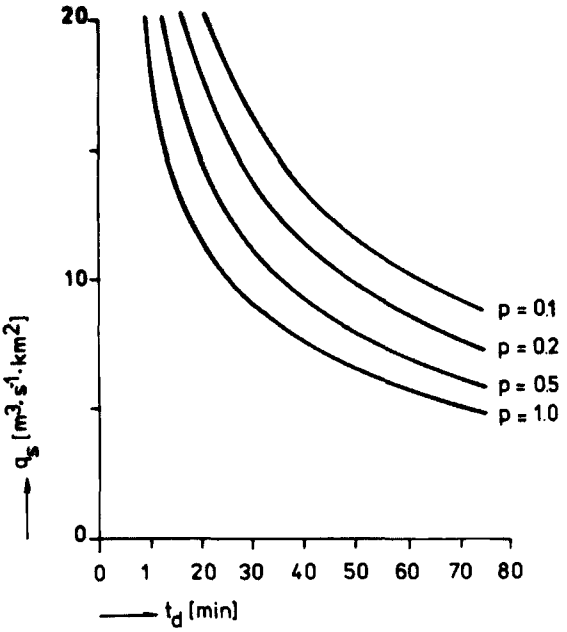


Fig.20 Rainfall rate duration curve.

1. Dub's formula:

$$q_S = \frac{3200}{(t_d + b)^{0.675}\,(150p)^n}, \tag{3.9}$$

where q_S – abundance of design rain ($1 \cdot s^{-1} \cdot ha^{-1}$)
t_d – time (duration) of design rain (min),
b – a parameter whose value depends on the time of design rain. It holds that: $b = 5/t_d$ for $t_d < 15$,
$b = 0$ for $15 < t_d < 60$,
$b = t_d/30$ for $t_d > 60$,
p – (0.1) periodicity of occurrence of design rain,
n – a parameter whose value depends on the sum of the average daily temperatures for the period from May 1 to Aug 31. It holds that:
$n = 0.25$ for $H_{S,d} > 8.5$,
$n = 0.30$ for $7.5 < H_{S,d} < 8.5$,
$n = 0.35$ for $H_{S,d} < 7.5$.

2. Urcikán's formula:

$$q_S = \frac{K}{t_d y + B} \tag{3.10}$$

Its parameters are indicated in Urcikán (1981).

3. Němec's formula:

$$i_{S,100} = \frac{(a \log t_d + b)\, n^c}{t_d}, \tag{3.11}$$

where $i_{S,100}$ – design rain intensity with a probability of incidence of once in 100 years, i.e., with a periodicity of $p = 0.01$ ($m \cdot s^{-1}$).

Its parameters are indicated in pertinent literature (DUB et al., 1969).

4. Čerkašín's formula:

$$q_{S,100} = 241.67 t_d^{-2/3} \tag{3.12}$$

where $q_{S,100}$ – abundance of design rain with a periodicity of $p = 0.01$ ($m^3 \cdot s^{-1} \cdot km^{-2}$),

t_d – time (duration) of design rain (min).

If there are not sufficient data for the calculation, the characteristics of the design rain can be arrived at indirectly, using Trupl's formula (HOLÝ et al., 1984):

$$H_S = u \log t_d + v \tag{3.13}$$

where H_S – ($= i_S t_d$) the amount of design rain (mm),

t_d – time (duration) of design rain (min),

u, v – parameters whose values depend on the periodicity of design rain (Table 4).

Table 4

Values of parameters u and v for equation 3.13 (HOLÝ, 1978)

Periodicity of design rain	u	v
0.01	36.0	−6.2
0.02	31.1	−4.6
0.05	24.4	−2.8
0.10	19.9	−1.4
0.20	15.7	0.0
0.50	11.4	+0.4
1.00	8.4	+0.8
2.00	6.2	+0.9

The p or t_d values of design rain are chosen according to the importance of the anti-erosion measure to be taken (for details see Section 5) or according to the so-called critical time (duration) of design rain ($t_{d,k}$).

The critical time of design rain means the duration of a design rain which brings about a maximum specific runoff (q_{max}) or maximum flow rate (Q_{max}). It holds that:

$$t_{d,k} \leqq t_k, \tag{3.14}$$

where t_k – time of concentration, i.e., the time taken by the water to arrive from the hydraulically most distant point of the watershed to the profile investigated (s).

3.2.1.2 Infiltration capacity of the soil

The soil's infiltration capacity is most accurately characterised by the so-called infiltration curve of the soil (Figure 21), i.e., the formula

$$v_{i,t} = f(t) \tag{3.15}$$

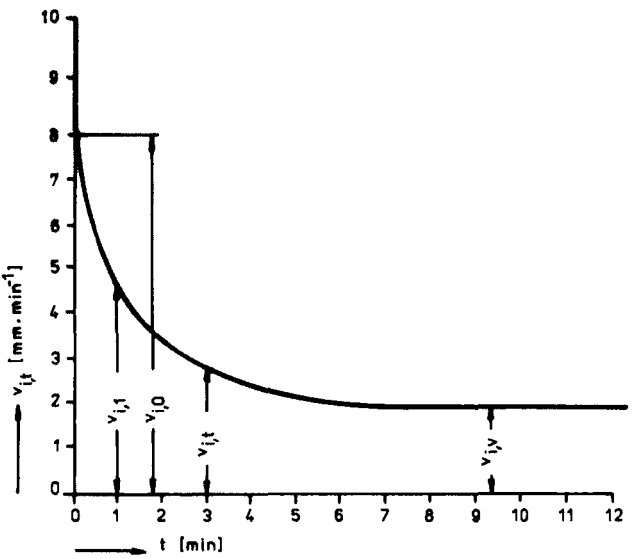


Fig.21 Infiltration curve of the soil.

where $v_{i,t}$ – intensity (rate) of the infiltration of water into the soil in time t from onset of infiltration (mm·min^{-1})

t – time from the onset of infiltration for which infiltration intensity is determined (min).

The data needed for the infiltration curve of the soil, i.e., the relationship in formula (3.15), are most frequently obtained through evaluating the results of a field infiltration trial with two concentric cylinders and an automatic dosing device (Fig. 22); the course of this curve is fairly accurately expressed by any of the following three empirical equations:

a) *Kostyak's equation:*

$$v_{i,t} = v_{i,1} t^{-\alpha}, \tag{3.16}$$

b) *Mezentsev's equation:*

$$v_{i,t} = v_{i,v} + (v_{i,1} + v_{i,v})\, t^{-\beta}, \tag{3.17}$$

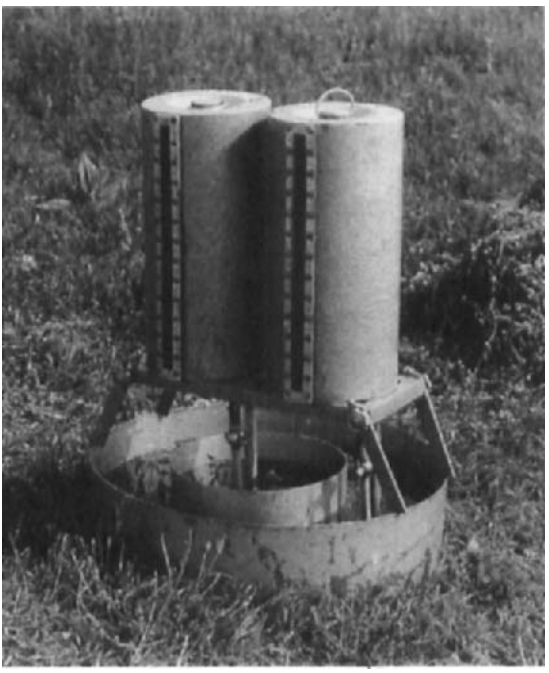


Fig.22 Apparatus to measure the infiltration capacity of the soil. (Photo J. Antal.)

c) *Horton's equation:*

$$v_{i,t} = v_{i,v} + (v_{i,0} - v_{i,v}) \exp(-\gamma t), \tag{3.18}$$

where $v_{i,t}$ – infiltration rate in time t ($m \cdot s^{-1}$, $mm \cdot min^{-1}$),
$v_{i,1}$ – initial infiltration rate for the Kostyak and Mezentsev equations, i.e. infiltration rate at the end of the first unit of time ($t = 1$) ($m \cdot s^{-2}$, $mm \cdot min^{-1}$),
$v_{i,0}$ – initial infiltration rate for the Horton equation, i.e., infiltration rate in time $t = 0$ ($m \cdot s^{-1}$, $mm \cdot min^{-1}$),
$v_{i,v}$ – stabilised (final) infiltration rate ($\doteq K_S$) ($m \cdot s^{-1}$, $mm \cdot min^{-1}$),
t – time from the onset of infiltration for which the infiltration rate is determined (s, min),
K_S – saturated hydraulic conductivity of the soil ($m \cdot s^{-1}$, $mm \cdot min^{-1}$),
α, β, γ – exponents obtained through the evaluation of the results of infiltration experiments.

When there are not sufficient data on the infiltration capacity of the soil investigated, the graphs in Fig. 23 can be used for determining the soil's infiltration capacity. These diagrams show the typical courses of the so-called cumulative infiltration curves, i.e., they express the dependence of the total amount of water infiltrated into the soil on different infiltration times and soil categories. It holds that:

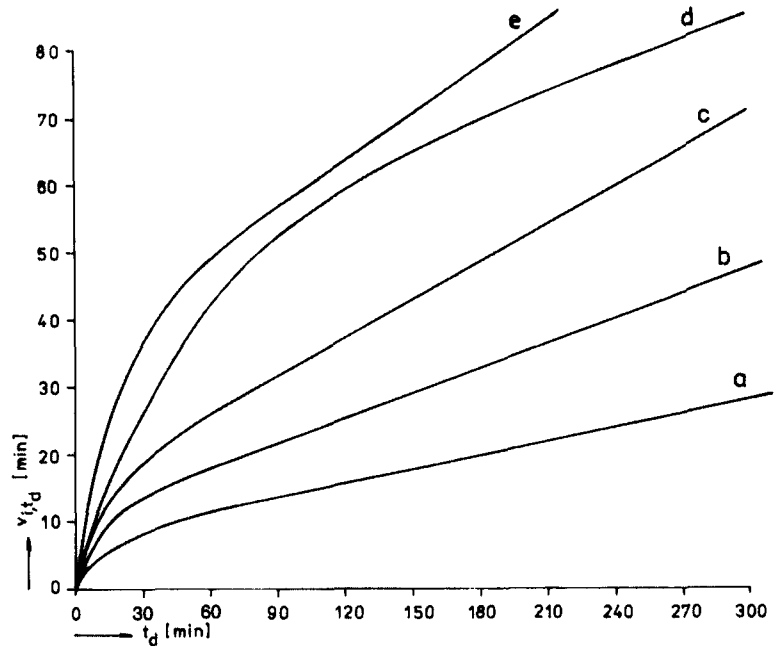


Fig.23 Characteristic course of the cumulative infiltration curves for the different soil categories a–e. (After HRÁDEK, 1981.)

$$V_{i,t} = \int_0^t v_{i,t}\,\mathrm{d}t, \tag{3.19}$$

where $V_{i,t}$ – amount of infiltrated water in time t from the onset of infiltration.

3.2.1.3 The rise of surface runoff of precipitation water

As to the relationship between the intensity of design rain (i_S) and the characteristics of the infiltration curve of the soil, three characteristic cases may arise, each with a different influence of changes in the time (duration) of design rain (t_d) on the rise and characteristics of surface runoff (Fig. 24):

1. if the intensity of design rain ($i_{S,1}$) equals or is lower than the stabilised infiltration rate ($v_{i,v}$), i.e., if it holds that

$$i_{S,1} \leqq v_{i,v}, \tag{3.20}$$

then no surface runoff of rainwater will occur, however long the rain may last (Fig. 24b);

2. if the intensity of design rain ($i_{S,2}$) equals or is higher than the initial infiltration rate ($v_{i,0}$ or $v_{i,1}$), i.e., if it holds that

$$i_{S,2} \geqq v_{i,0} \text{ or } v_{i,1}, \tag{3.21}$$

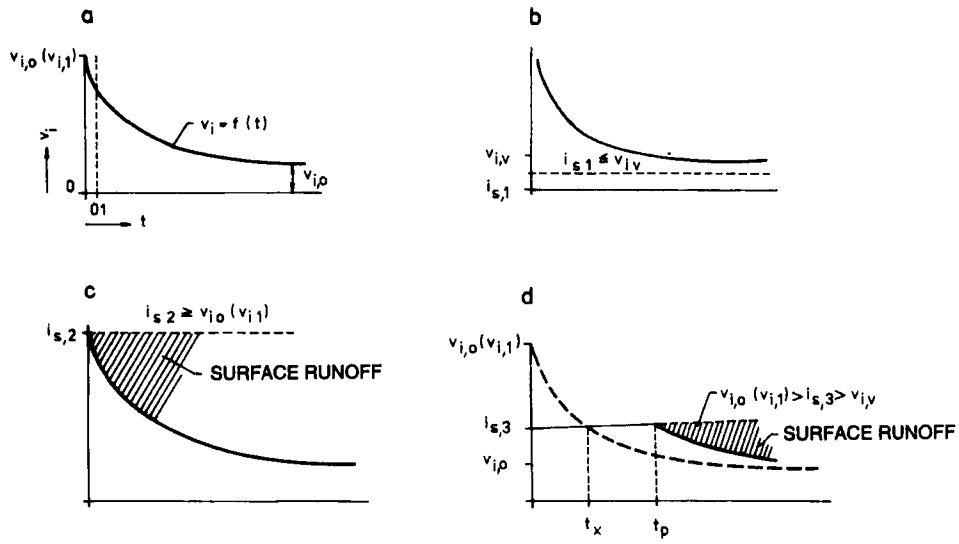


Fig.24 Effect of the intensity and duration of design rain on the rise of surface runoff. (After ANTAL, 1984.)

then, not taking into account the surface retention of rain water, surface runoff starts immediately with the onset of the rain (Fig. 24c);

3. if the intensity of design rain ($i_{S,3}$) is higher than the stabilised infiltration rate ($v_{i,v}$) yet lower than the initial infiltration rate ($v_{i,0}$ or $v_{i,1}$), i.e., if it holds that

$$v_{i,0} \text{ or } v_{i,1} > i_{S,3} > v_{i,v}, \tag{3.22}$$

then surface runoff will only set in if the time of substitute rain is longer than the time to the onset of the formation of pools (t_p).

The time to the onset of the formation of pools (on flat ground) or to the onset of surface runoff (on a slope) can be determined either by calculation or graphically.

When calculating the t_p it is assumed that the formation of pools starts when the amount of water infiltrated into the soil reaches a level corresponding to that on the theoretical infiltration curve of the soil, i.e., when area F_2 in Fig. 24d equals the value of area F_1; this can be described by the following equation:

$$\int_0^{t_x} v_{i,t}\, dt = i_{S,3} t_p, \tag{3.23}$$

where t_x – time from the onset of infiltration during which the line representing the intensity of rain crosses the theoretical infiltration curve, i.e. the time in which it holds that $i_{S,3} = v_{i,t_x}$ (min).

The procedure of calculating the t_x for the case of the Mezentsev equation is shown in formulae 3.24a and 3.24b:

$$i_{S,3} = v_{i,t_x} = v_{i,v} + (v_{i,1} - v_{i,v})\, t_x^{-\beta} \tag{3.24a}$$

$$t_x = \left(\frac{v_{i,1} - v_{i,v}}{i_{S,3} - v_{i,v}}\right)^{1/\beta}. \tag{3.24b}$$

A similar procedure can be used in calculating the t_x for cases in which the course of the infiltration curve of the soil is expressed by other equations.

3.2.1.4 Characteristics of surface runoff

To be able to design effective erosion-control measures, the characteristics of surface runoff should be known, particularly the following four:

1. volume of surface runoff from the design rain in question W (m^3),
2. the height of surface runoff $H_{Q,p}$ (mm),
3. flow for N years or design flow (Q_N) ($m^3 \cdot s^{-1}$),
4. the hydrogram of the flood wave induced by the design rain considered, i.e., $Q_p = f(t)$.

Erosion-control measures are usually prepared for areas where no direct hydrological observations are performed; hence, the above-mentioned characteristics have to be determined indirectly, usually by calculation.

For erosion-control purposes, the characteristics of surface runoff are most frequently calculated in the following three ways, using:

- intensity-type formulae (the *"rational method"*),
- a combination of formulae of genetic and intensity types (*Hrádek's method*),
- runoff curves (the *CN method*).

The rational method for the calculation of Q_{100} is expressed by an intensity-type formula in either of the following two forms:

$$q_{100} = q_{S,100}\varphi_N \tag{3.25}$$

or

$$Q_{100} = q_{100}S_p = q_{S,100}\varphi_N S_p, \tag{3.26}$$

where q_{100} – maximum specific surface runoff with a recurrence time of $N = 100$ years or with a periodicity of $p = 0.01$ ($m^3 \cdot s^{-1} \cdot km^{-2}$),

Q_{100} – maximum flow rate for $N = 100$ or $p = 0.01$ ($m^3 \cdot s^{-1}$),

$q_{S,100}$ – abundance of design rain at $N = 100$ or at $p = 0.01$ ($m^2 \cdot s^{-1} \cdot km^{-2}$),

φ_N – ($= q_{100}/q_{S,100}$) simulated, (peak) coefficient of surface runoff applying to cases in which $t_{d,k} = t_k$,

S_p – watershed area (catchment area) (km^2).

The rational method was developed on the following simplifying assumptions:

1. The length (duration) of design rain (t_d) equals the critical length of design rain ($t_{d,k}$) and the critical length of substitute rain equals the concentration time (t_k), i.e.,

$$t_d = t_{d,k} = t_k. \tag{3.27}$$

2. The rain stays in the whole watershed investigated.

3. The maximum runoff (Q_{100}) or maximum specific surface runoff (q_{100}) is caused by rain of an abundance of $q_{S,100}$ or an intensity of $i_{S,100}$.

Using a triangular schematisation of the actual form of the hydrogram of the flood wave (Fig. 25) under the same conditions as for the use of the intensity formula for calculating the q_{100} or Q_{100}, HRÁDEK (1981) constructed his "genetic formula of intensity type":

$$q_{100} = \frac{2\varphi q_{s,100} t_{d,k}}{t_k(1 + n_h)} \tag{3.28}$$

or

$$Q_{100} = \frac{2\varphi q_{S,100} t_{d,k} S_p}{t_k(1 + n_h)}, \tag{3.29}$$

where φ – $(= W_{100}/S_{100})$ volume coefficient of surface runoff,

n_h – $(= t_p/t_v)$ coefficient of the form of the hydrogram of the flood wave,

W_{100} – $(= Q_{100} t_c/2)$ volume of the flood wave caused by design rain of a duration of t_d $(t_d = t_{d,k})$ and of an abundance of $q_{S,100}$ (m^3),

S_{100} – $(= q_{S,100} t_{d,k} S_p)$ amount (volume) of the design rain considered (m^3),

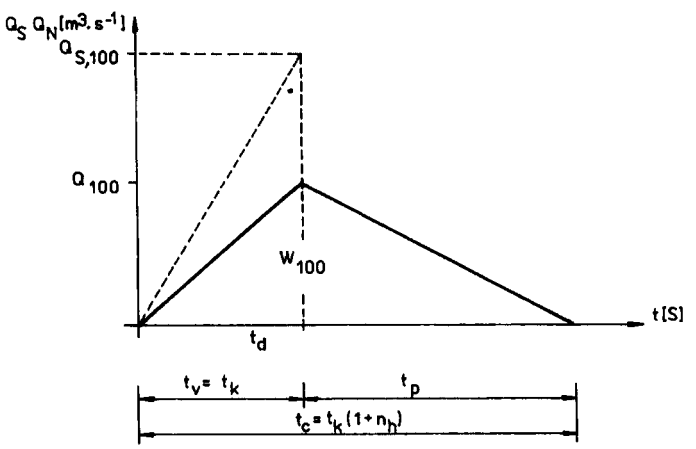


Fig.25 Triangular schematisation of the shape of a flood-wave hydrograph.

t_c – $[= t_v + t_p = t_k(1 + n_h)]$ time (duration) of the flood wave (s),
t_v – $(= t_k)$ rising time of the flood wave (s),
t_p – receding time of the flood wave (s).

For the triangular form of the flood wave and for $t_{d,k} = t_k$ it holds that:

$$\varphi_N = \frac{2\varphi}{1 + n_h}. \tag{3.30}$$

Using equation (3.30), genetic formulae of intensity type (3.28) and (3.29) can be rearranged to the form of

$$q_{100} = \varphi_N q_{S,100} \frac{t_{d,k}}{t_k} \tag{3.31}$$

$$Q_{100} = \varphi_N q_{S,100} S_p \frac{t_{d,k}}{t_k}. \tag{3.32}$$

The data to be used for the calculation of q_{100} or Q_{100}, using the genetic formula of intensity type – which, if $t_{d,k} = t_k$, becomes an intensity-type formula – are obtained as follows:

a) to obtain the value of $q_{S,100}$, use any of the procedures indicated in the description of the so-called design rain intensity lines (e.g., the Dub formula, the Cherkashin formula, Reinhold's method, etc.); the best procedures are those that directly express the relationships

$$q_{S,100} = f(t_d), \quad \text{or} \quad i_{S,100} = f(t_d);$$

b) determine the value of n_h according to the Hrádek formula (HRÁDEK, 1981):

$$n_h = \frac{4}{3}(q_{S,100}\varphi)^{0.25}; \tag{3.33}$$

c) to obtain the value of φ, use the equation

$$\varphi = \frac{W_{100}}{S_{100}} = \frac{H_{O,p,t_d} S_p}{H_{S,t_d} S_p} = \frac{H_{O,p,t_d}}{H_{S,t_d}} = \frac{H_{S,t_d} - V_{i,t_d} - H_R}{H_{S,t_d}}, \tag{3.34}$$

issuing from the basic formula for the calculation of the runoff volume coefficient in the form of $\varphi = W_{100}/S_{100}$ and using the equation of the hydrological balance of soil surface during the course of the design rain – formula (3.35) below (Fig. 26):

$$H_{O,p,t_d} = H_{S,t_d} - V_{i,t_d} - H_R, \tag{3.35}$$

where φ – runoff volume coefficient for the design rain considered,
H_{O,p,t_d} – $(= W_{100}/S_p)$ level of surface runoff from the design rain considered (m),
H_{S,t_d} – $(= i_{S,100} t_d)$ level of the design rain considered (m),

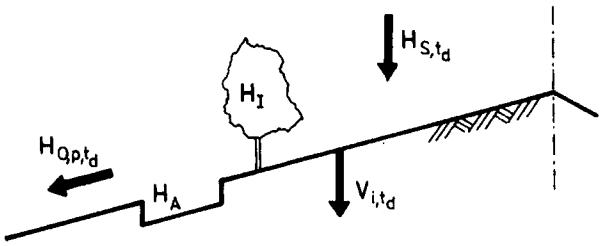


Fig.26 Components of the hydrologic balance of the soil surface.

V_{i,t_d} – ($= \int_0^{t_d} v_{i,t}\,dt$) level of water infiltrated into the soil during the course of the design rain considered (m),
H_R – ($= H_A + H_I$) total level of retention of precipitation water on the soil surface (m) (Tab. 5),
H_A – level of accumulated precipitation water on the soil surface (m),
H_I – level of interception of precipitation water by the plant cover of the soil (m);

d) to calculate the value of t_k, use for example the equation

$$t_k = \frac{l_{sv}}{v_{sv}} + \frac{l_a}{v_a}, \tag{3.36}$$

where t_k – time of concentration (s),
l_{sv} – average length of the surface runoff of water down the slope (m),
l_a – length of the surface runoff of water down the valley line (water course) (m),
v_{sv} – average speed at which the water moves down the slope ($m \cdot s^{-1}$) (Tab. 6),
v_a – average speed at which water moves down the valley line (water course) ($m \cdot s^{-1}$) (Tab. 7);

Table 5

Retention of precipitation water on soil surface (HRÁDEK, 1962)

Group of soils	Nature of soil surface	H_R (mm)	average H_R (mm)
A	Bare surface	2– 4	3
B	Fields	6– 8	7
C	Meadows	10–18	14
D	Conifer stands with a layer of humus	12–25	19
E	Mature hardwood stands	6–12	9
F	Young forests	6–10	8
G	Peat bogs, wetlands	30–36	33
H	Moss	40–45	43

e) the best way of determining the value of $t_{d,k}$ is to evaluate the relationship $q_{100} = f(t_d)$ which, in turn, is determined by successive calculations of q_{100} values for different design rains of durations of $t_d = t_k$ (Tab. 8). The maximum value of function $q_{100} = f(t_d)$ specifies the values of both q_{100} and $t_{d,k}$ for the watershed investigated (Fig. 27).

Table 6

Average velocity of the movement of water down slope (HRÁDEK, 1982)

Average gradient of slope (%)	1	3	5	10	15	20
	v_{sv} (m·s^{-1})					
Fields	0.05	0.11	0.17	0.32	0.40	0.62
Meadows, afforested watershed (L < 50%)	0.03	0.07	0.10	0.18	0.25	0.33
Afforested watershed ($L \geq 50\%$)	0.02	0.05	0.07	0.12	0.17	0.22

Table 7

Average velocity of the movement of water along a valley line (in water course) (HRÁDEK, 1982)

Average gradient (%)	1	3	5	10	15	20
v_u (m·s^{-1})	0.05	0.11	0.17	0.32	0.40	0.62

Table 8

A survey of the calculation of $t_{d,k}$ for a watershed with the following characteristics:

$t_k = 105$ (min)
$q_{S,100} = 241.67 t_d^{-2/3}$ (m^3·s^{-1}·km^{-2})
soil category IV.
$H_R = 7$ (mm) (ANTAL, 1985)

Parameter	t_d (min)						
	105	90	75	60	45	30	15
$q_{S,100}$ (m^3·s^{-1}·km^{-2})	10.86	12.03	13.59	15.77	19.10	25.03	39.73
H_{S,t_d} (mm)	68.42	64.96	61.16	56.77	51.57	45.05	35.76
V_{i,t_d} (mm)	56.50	52.00	47.00	41.50	35.00	27.50	17.50
H_R				7.00			
φ	0.07	0.09	0.12	0.15	0.19	0.23	0.31
n_h	1.25	1.36	1.51	1.65	1.84	2.07	2.50
φ_N	0.06	0.08	0.10	0.11	0.13	0.15	0.18
q_{100} (m^3·s^{-1}·km^{-2})	0.65	0.82	0.97	0.99	1.06	1.07	1.02

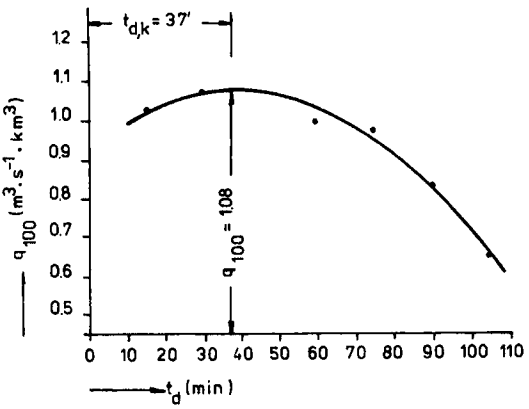


Fig.27 Determination of the critical time of duration of design rain.

If there are not sufficient data for the calculation, the approximate value of the runoff volume coefficient φ can be assessed according to the Härtel formula, modified by ČERMÁK (HOLÝ, 1978):

$$\varphi = n_1 n_2 n_3 n_4, \tag{3.37}$$

where $n_1 - n_4$ – partial characteristics of the watershed influencing the φ value (see Tab. 30); the approximate value of the simulated surface runoff coefficient (φ_N) can be read from Table 9.

The flow rate Q_N with a required periodicity of $p = 1/N$ can be computed by the formula

$$Q_N = a_N Q_{100}, \tag{3.38}$$

where a_N – reduction coefficient for an N-year flow rate (Tab. 10).

Table 9

Orientational data on the design coefficient of runoff (φ_N)

Soil \ I (%)	2	4	6	8	10	12	14	16	18	20
Light	0.08	0.11	0.14	0.15	0.17	0.18	0.20	0.21	0.22	0.23
Medium	0.10	0.14	0.17	0.20	0.22	0.24	0.25	0.26	0.27	0.28
Heavy	0.12	0.17	0.22	0.24	0.28	0.29	0.31	0.34	0.35	0.36
	$I < 5\%$				$I \geq 5\%$					
Forests	0.05				0.10					
Orchards	0.15				0.20					
Inside villages and towns	0.30				0.40					

Table 10

Values of the reduction coefficient a_N for the calculation of Q_N

Watershed	a_N					
	a_1	a_2	a_5	a_{10}	a_{20}	a_{50}
Lowland						
No forest	0.14	0.22	0.34	0.45	0.54	0.76
Partial afforestation	0.18	0.29	0.44	0.55	0.67	0.84
Slightly sloping						
No forest	0.10	0.14	0.23	0.33	0.47	0.73
Partial afforestation	0.14	0.21	0.33	0.45	0.60	0.81
Sloping						
No forest	0.06	0.08	0.13	0.21	0.34	0.62
Partial afforestation	0.10	0.15	0.23	0.33	0.47	0.70

Another method used to calculate the characteristics of surface runoff is the CN method, or the method of runoff curves.

The CN method was developed from long-term observations and numerous analyses of the precipitation-runoff relationships mainly in farmed watersheds. Hence, the CN method (more than any other) takes into account the effect of farming on surface runoff characteristics (Tab. 11). The following key factors are involved in the runoff calculations by the CN method:

1. the height of the design (and also actual) rain – $H_{S,N}$, determined, e.g., by equation (3.13);
2. the infiltration and drainage characteristics of the soils in the watershed (Tab. 12);
3. hydrological characteristics of the soil cover for:
   a) field crop rotations (Tab. 13),
   b) permanent grassland,
   c) pastures (Tab. 14),
   d) tree stands (Tab. 15);
4. the position of soil moisture (only for the calculation of characteristics for an actual rain), characterised by the index of previous precipitation – IPS (Tab. 16);
5. the use of the land, the orientation of soil cultivation (direction of furrows) and the effect of anti-erosion infiltration ditches and channel terraces (if there are any) (Tab. 17).

Using these characteristics of the watershed investigated, the corresponding CN value can be seen from Table 18 and the surface runoff characteristics can be calculated by the following equations:

$$H_{O,p,N} = \frac{(H_{S,N} - 0.2H_{R,p})^2}{H_{S,N} + 0.8H_{R,p}}, \tag{3.39}$$

Table 11

Comparison of different methods for the calculation of Q_{max}

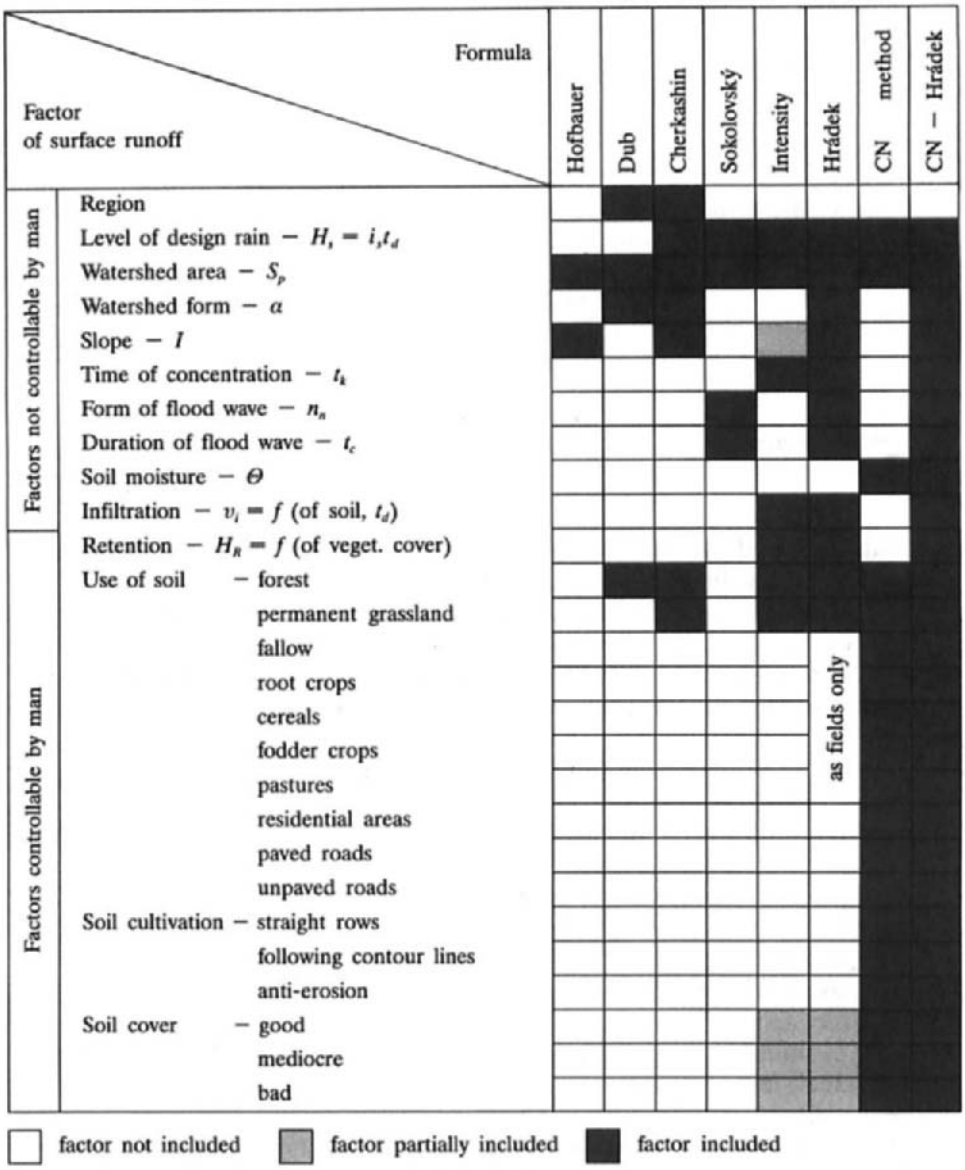


Table 12

Hydrologic categories of the soil for the CN method (VEN TE CHOW, 1964)

Soil category	Infiltration and drainage characteristics of soils	Characteristic soil types
A	Soils with a high infiltration capacity even at a full water saturation	Deep sands and gravels
B	Soils with a medium infiltration capacity even at a full water saturation and with a good drainage function	Medium-deep to deep sands and loamy sands
C	Soils with a low infiltration capacity at full water saturation and with a poor drainage fuction	Shallow clay loams and clays
D	Soils with a very poor infiltration capacity and with no drainage function	Clays or soils with otherwise limited drainage and infiltration capacities

Table 13

Hydrologic properties of field crop rotations (VEN TE CHOW, 1964)

Proportion of root crops in the field crop rotation	Hydrologic properties
< 50%	good
> 50%	poor

Table 14

Hydrologic properties of natural grassland (VEN TE CHOW, 1964)

% cover of soil by grass	Yield converted to dry matter ($t \cdot ha^{-1}$)		
	< 1.0	1−3	> 3
< 50	poor	poor	average
50−75	poor +	average	average +
> 75	average	average +	good

Note: the + sign means that the resultant CN value is obtained through extrapolation between the CN value indicated and the first subsequent **better** CN value. For example, a pasture on soil category "C" denoted as "poor +" would have a resultant value (Tab. 21) of CN = (79 + 74) : 2 = 76.5.

$$W_N = 1000 H_{O,p,} S_p, \tag{3.40}$$

$$Q_N = \frac{H_{O,p,N} S_p}{k t_v}, \tag{3.41}$$

Table 15

Hydrologic properties of tree stands (VEN TE CHOW, 1964)

Classification of tree stands	Characteristics of soil surface cover under tree stands
bad	no shed leaves and branches on soil surface, no shrubs
average	soil surface partly covered with material shed from trees and by shrubs
good	soil surface completely covered with material shed from trees and by shrubs

Table 16

Index of previous precipitation – IPS (VEN TE CHOW, 1964)

IPS	Total precipitation for the past 5 days	
	outside growing season*	in growing season
I	< 13 (mm)	< 36 (mm)
II	13–28 (mm)	36–53 (mm)
III	> 28 (mm)	> 53 (mm)

* while the soil is not frozen or covered with snow

where $H_{O,p,N}$ – surface runoff from the design rain with a periodicity of $p = 1/N$ (mm),

$H_{S,N}$ – $(= i_{S,N} t_{d,k})$ the level of the design rain considered (mm),

$H_{R,p}$ – potential retention in the watershed (mm),

W_N – volume of the flood wave produced by the design rain (m^3),

S_p – size of the watershed (km^2),

k – correction coefficient (= from 3.9 for mountain regions to 7.7 for lowlands, usually $k = 4.8 - 5.3$),

t_v – length of the ascending stage of the flood wave (h).

The value of potential retention in the watershed $H_{R,p}$ is calculated by the formula

$$H_{R,p} = 25.4\left(\frac{100}{CN} - 10\right), \tag{3.42}$$

where CN – number of the "runoff curve", determined according to the data in Tab. 16 and / or with the use of Fig. 28 (for IPS = II).

The value of t_v for equation (3.41) is computed as follows:

$$t_v = \frac{(3.28 l_{\max})^{0.8}(0.04 H_{R,p} + 1)^{0.7}}{1900 I^{0.5}}, \tag{3.43}$$

Table 17

Average annual CN values for IPS = II (VEN TE CHOW, 1964)

Use of land	Hydrologic characteristics					
	Soil cultivation	Soil cover	Soil properties			
			A	B	C	D
Fallow			77	86	91	94
Wide-row crops	in straight rows	bad	72	81	88	91
	in straight rows	good	67	78	85	89
	along contour lines	bad	70	79	84	88
	along contour lines	good	65	75	82	86
	terracing	bad	66	74	80	82
	terracing	good	62	71	78	81
Narrow-row crops	in straight rows	bad	65	76	84	88
	in straight rows	good	63	75	83	87
	along contour lines	bad	63	74	82	85
	along contour lines	good	61	73	81	84
	terracing	bad	61	72	79	82
	terracing	good	59	70	78	81
Fodder crops on arable land, temporary meadows	in straight rows	bad	66	77	85	89
	in straight rows	good	58	72	81	85
	along contour lines	bad	64	75	83	85
	along contour lines	good	55	69	78	83
	terracing	bad	63	73	80	83
	terracing	good	51	67	76	80
Pastures		bad	68	79	86	89
		good	49	69	79	84
		bad	39	61	74	80
	along contour lines	good	47	67	81	88
	along contour lines	bad	25	59	75	83
	along contour lines	good	6	35	70	79
Meadows, permanent grassland		good	30	58	71	78
Forest		bad	45	66	77	83
		average	36	60	73	79
		good	25	55	70	77
Inside villages and towns			59	74	82	86
Non-paved roads			72	82	87	89
Paved roads			74	84	90	92

Table 18

Calculation of $H_{O,p}$ for IPS = I and H_S = 109 mm for a watershed (the watershed's characteristics are shown in the table) (ANTAL, 1985)

Use of land	Soil cultivation	State of soil cover	Area (ha)	CN for		$H_{O,p}$ (mm)	(4) × (7)
				IPS-II	IPS-I		
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
Cereals	along slope line	good	469.45	85	70	38.8	18 214.7
Clovers	along slope line	good	218.54	81	64	29.0	6 337.7
Pastures		average	84.99	79	62	26.0	2 209.7
Forests		good	138.81	70	51	11.9	1 651.8
Paved roads			33.18	90	78	53.9	1 788.4
			∑944.97				30 202.3

$$H_{O,p} = \frac{\sum(8)}{\sum(4)} = \frac{30\,202.3}{944.97} = 32 \text{ mm}$$

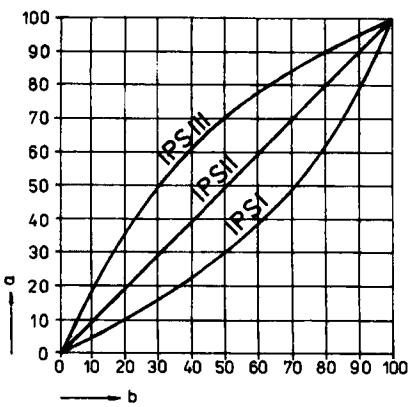


Fig.28 Relationships of the runoff curves IPS I, IPS II and IPS III (a) and runoff curve IPS II (b).

where l_{max} – distance from the hydraulically most distant point in the watershed to the profile studied (m),

I – average gradient of slope in the watershed studied.

Detailed analysis of equation (3.41) can demonstrate that if data in km², mm and h are substituted for S_p, $H_{Q,p,N}$, respectively, it will hold that

$$k = 1.8(1 + n_h), \tag{3.44}$$

as shown by ANTAL (1985).

The procedure of calculating $H_{O,p}$ for watersheds with different uses of land is shown in Tab. 18.

The hydrogram of the flood wave induced by the contemplated design rain (such a hydrogram is needed for some anti-erosion measures, e.g., combined erosion-control ditches) can only be constructed if data are available for calculating surface runoff characteristics by the intensity-type genetic formula or by the CN method, i.e., for calculating at least the Q_{100}, t_v and n_h: these data would be sufficient for constructing the schematised hydrogram of the flood wave of triangular shape (see Fig. 25).

3.2.2 Erosion potential of water and wind

The erosion potential of water and wind, sometimes also called erosivity, i.e., the ability of water and wind to cause soil erosion, is a function of:

- characteristics of rain,
- characteristics of surface runoff,
- characteristics of wind close to the soil surface.

The key characteristics of rain on which its erosivity depends include:

- the size of rain drops – d (mm),
- the velocity of incidence of rain drops onto the soil surface – v_t ($m \cdot s^{-1}$),
- the kinetic energy of rain – E ($J \cdot m^{-2}$).

The experimentally verified relationships between rain intensity and the average size of rain drops (Fig. 29) and / or between the average size of rain drops and the velocity of their incidence on the soil surface (Fig. 30) were used by Wischmeier

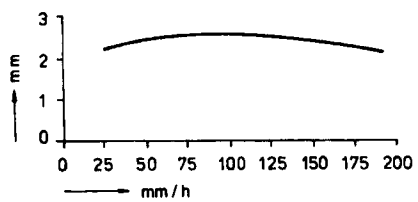


Fig.29 Effect of rain intensity (in $mm \cdot h^{-1}$) on the size of rain drops (in mm). (After HUDSON, 1971.)

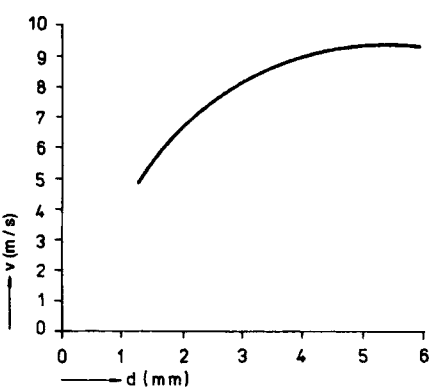


Fig.30 Effect of the size of rain drops (in mm) on the rate of their incidence on a soil surface (in $m \cdot sec^{-1}$). (After HUDSON, 1971.)

and Smith (1958) to develop their equation of the kinetic energy of the rain segment:

$$E_n = (206 + 87 \log i_{S,n}) H_{S,n}, \tag{3.45}$$

where E_n – kinetic energy of the n^{th} rain segment ($J \cdot m^{-2}$),
$i_{S,n}$ – average rain intensity in the n^{th} rain segment ($cm \cdot h^{-1}$),
$H_{S,n}$ – level (height) of rain in the n^{th} rain segment (cm).

The total kinetic energy of the given rain is calculated as the sum of the E_n of the separate rain segments. An example of calculation of the E for the rain represented in Fig. 31 is shown in Tab. 19; it holds that

$$E = \sum E_n. \tag{3.46}$$

As shown by the analysis of experimental data on the water erosion of the soil, the relationship between the characteristics of rain and the intensity of water

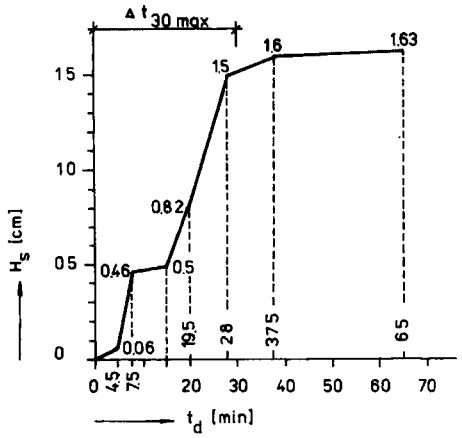


Fig.31 Ombrographic record of rain for the calculation of its kinetic energy.

Table 19

Procedure for the calculation of E for rain shown in Figure 31

Number of rain division	t min	Δt min	H_S cm	ΔH_S cm	i_S		E ($J \cdot m^{-2}$)	
					$mm \cdot h^{-1}$	$cm \cdot h^{-1}$	per 1 cm H_S	per ΔH_S
1	4.5	4.5	0.06	0.06	8.0	0.80	198	12
2	7.5	3.0	0.46	0.40	80.0	8.00	285	114
3	15.0	7.5	0.50	0.04	3.2	0.32	163	7
4	19.5	4.5	0.83	0.32	42.7	4.27	261	84
5	28.5	9.0	1.50	0.68	45.3	4.53	263	179
6	37.5	9.0	1.60	0.10	6.7	0.67	191	19
7	65.0	27.5	1.63	0.03	0.7	0.07	106	3

erosion is best expressed (hence the erosivity of rain is best characterised) by the so-called rain erosivity factor (also referred to as the rain factor – R). The rain factor is defined as the product of the kinetic energy of erosion-active (erosive) rain (E) and its highest 30-minute intensity (I_{30}), i.e. (WISCHMEIER and SMITH, 1958):

$$R = EI_{30}10^{-2}, \tag{3.47}$$

where R – rain erosivity factor ($MJ \cdot ha^{-1}$),
E – kinetic energy of rain ($J \cdot m^{-2}$),
I_{30} – maximum intensity of the given rain during a period of 30 minutes (see Fig. 33) ($cm \cdot h^{-1}$).

The R values calculated for the different erosive rains can be subjected to statistical analysis to obtain the daily, weekly, monthly, annual and N-year R-factor values as well as the distribution of the effect of the N factor during the year (Fig. 32). The method of calculation of the R factor is shown in section 4.3.1.

In calculating the kinetic energy by equations (3.45) and (3.46), HUDSON (1971) does not take into account the rain segments in which $i_{S,n} < 25\ mm \cdot h^{-1}$: for the R value calculated in this way Hudson uses the expression KE > 25.

A comparison of the calculations of E for actual rains based on the above-mentioned criteria of the erosive rain, or its erosion-active component, is shown in Tab. 20.

To evaluate the erosion potential of precipitation water running off down the surface of the soil, the following two parameters should be known:

– velocity of the precipitation water running off down the surface which can be calculated, for example, by the following formula:

$$v_x = (ai_S\varphi x)^{0.5}; \tag{3.48}$$

– tangential stress of the water running off down the surface; this can be calculated, for example, as follows:

$$\tau_x = \gamma_v I \left(\frac{\varphi i_s x}{a}\right)^{0.5}, \tag{3.49}$$

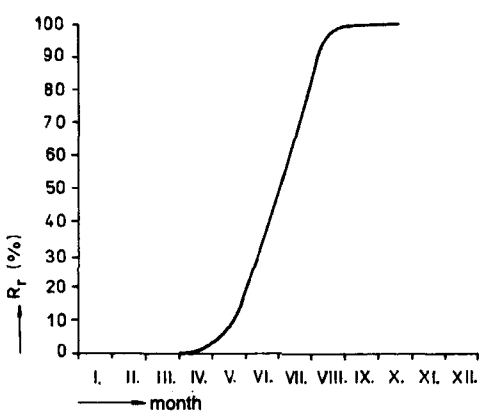


Fig.32 Distribution of the effect of the rain factor during the year. (After PASÁK et al., 1983.)

Table 20

Calculation of the *R* value by different methods (characteristics of rain as recorded at Nitra, Slovakia, on June 21, 1973)

Number of rain division	t min	Δt min	H_S cm	ΔH_S cm	i_S cm·h^{-1}	E_n (J·m^{-2}) after		
						W.-S.	Hudson	Šabata
1	16	16	0.70	0.7	2.625	169.8	169.8	
2	26	10	0.96	0.26	1.560	57.9		
3	46	20	2.30	1.34	4.020	346.5	346.5	346.5
4	52	6	2.44	0.14	1.400	30.6		
5	70	18	2.62	0.18	0.600	33.6		
6	95	25	2.70	0.08	0.192	12.5		
7	355	260	2.98	0.28	0.065	34.4		
						$\sum E_n$ = 685.3	516.3	346.5

where v_x – velocity of the water running down the soil surface at distance x from the watershed divide (m·s^{-1}),

τ_x – tangential stress of the water running down the surface at distance x from the watershed divide (N·m^{-2}),

a – coefficient characterising the condition and inclination of the soil surface.

The procedure for deriving equations (3.48) and (3.49) is shown in section 4.1. To assess the erosivity of wind, it is necessary to know the following three key parameters:

- wind speed in the closest proximity to the soil surface (m·s^{-1}),
- time of exposure to the wind (h),
- wind direction in relation to the soil surface ("windward exposure") (k_n) for which it holds that:

$$k_n = \cos \alpha, \tag{3.50}$$

where α – angle of incidence of the air current onto the soil surface (°).

The wind of the highest erosivity will have a k_n equal to 1; such a wind will blow parallel to the soil surface ($\alpha = 0°$). On the other hand, at a $k_n = 0$, i.e. at ($\alpha = 90°$), the wind has the lowest erosivity.

3.2.3 Erodibility of the soil

The erodibility of the soil, i.e., the soil's liability to erosion (or conversely, the resistance of the soil to the effect of erosion factors), is a function of the soil's properties, particularly its physical characteristics such as texture, permeability, moisture content and others. The erodibility of the soil is also influenced by the humus content in the soil, soil structure and the saturation of the soil's sorption complex with basic cations.

Considering the action of the main erosion factors, the erodibility of the soil is assessed by its ability to resist:

- the erosivity of rain,
- the kinetic energy of water running off down the soil surface,
- the kinetic energy of wind.

The ability of the soil to resist the erosivity of rain is expressed by what is called the soil erodibility factor K which can be experimentally determined in a unit plot (see Section 4.3), using the formula:

$$K = \frac{S_p}{R}, \tag{3.51}$$

where K – the soil erodibility factor ($t \cdot MJ^{-1}$),
R – erosivity of the rain in question ($MJ \cdot ha^{-1}$),
S_p – loss of soil (soil erosion) from unit plot, as caused by the rain in question ($t \cdot ha^{-1}$).

The soil's ability to resist the kinetic energy of the precipitation water running away down the soil surface and the kinetic energy of surface wind is assessed using the following three parameters (Tab. 21):

Table 21

Orientational values of v_k and τ_k (DÝROVÁ and ŠOUKALOVÁ, 1978)

Soil	$I < 10\%$		$I > 10\%$	
	v_k ($m \cdot s^{-1}$)	τ_k ($N \cdot m^{-2}$)	v_k ($m \cdot s^{-1}$)	τ_k ($N \cdot m^{-2}$)
Clay loam	0.318	4.14	0.245	2.46
Loam	0.322	4.24	0.248	2.52
Sandy loam	0.322	4.24	0.248	2.52
Loamy sand	0.343	4.82	0.264	2.85
Sand	0.397	6.45	0.305	3.81

- critical velocity of precipitation water running down the surface (v_k),
- critical shear stress of precipitation water running down the surface (τ_k),
- critical velocity of surface wind ($v_{v,k}$);

while it holds that the higher the v_k, τ_k and $v_{v,k}$ values of the soil the higher the resistance to the action of erosion factors, i.e. the lower the soil's erodibility.

Soil characteristics are closely related with the soil type: this fact allows us to arrange the soil types in an approximate order based on their erodibility. For example, according to HOLÝ (1978) the chernozem soil has the highest erosion resistance, followed by the brown earth; podzolic soil is the most susceptible to erosion under otherwise equal conditions.

3.2.4 Effect of other erosion factors on erosion intensity

In addition to the erosivity of rain, the kinetic energy of water running down the soil surface, the kinetic energy of surface wind and the erodibility of the soil (all these are primarily functions of the climatic, hydrological, soil and geological conditions), there are a number of other factors that contribute to soil erosion: these include the topographic factors, vegetation factors and the economic and technical conditions of the region in question.

These factors are very important for erosion control because man has means to modify them. Man can alter the plant life in the region and apply different patterns of economic and technical management; even the topographic conditions may be partially modified by human activities. This in turn may also influence the hydrological conditions. What cannot be controlled by man are the soil conditions, the geology and climate of the region.

3.2.4.1 Effect of topographic conditions on soil erosion

It follows from the theoretical analysis of the mechanism of water erosion of the soil, confirmed by the results of field and laboratory experiments, that slope inclination (gradient) and slope length are the key topographic characteristics of the region as regards their influence on erosion. Though this effect on the rise and course of the erosion process may be reduced by other erosion factors (e.g., through a change or modification of the vegetation conditions and the economic and technical factors), it can never be entirely eliminated.

The key position of the gradient and length of slope among the factors influencing water erosion ensues from the fact that these two traits, in addition to the characteristics of the soil and rain, exert the greatest influence on the velocity, tangential stress and energy of the precipitation water running down the soil surface (see Section 4.1).

Some experimentally obtained data on the relationship between slope gradient and length and the intensity of the water erosion of the soil are shown in Tab. 22 and in Fig. 33. It clearly follows from these data that the intensity increases rapidly with higher values of these parameters, and their demonstrated key influence on the rise and course of the water erosion of the soil encouraged researchers throughout the world to seek the values of the critical gradient (Tab. 23) and length at which erosion intensity exceeds the maximum admissible intensity level or at which areal erosion passes into degree I linear erosion, i.e., rill erosion.

Under real conditions, the actual values of these critical parameters also depend on other erosion factors, particularly the climate, soil and vegetation in the region investigated.

The methods of determining the critical length of slope are shown in Section 4.1. If the slope gradient is to be changed, complex and costly technical efforts are always required (e.g., terracing); hence, in erosion-control measures, the critical gradient of slope is always considered in combination with determining the critical slope length.

The effect of slope gradient, i.e., the effect of the shape of the slope's cross-section (and also the influence of changes in soil characteristics, vegetation cover and erosion-control measures on the slope) is now most frequently expressed as

Table 22

Effect of slope length and gradient on the removal of soil (ZACHAR et al., 1984)

Site	Author	Slope length (m)	Slope gradient (°)	Amount of soil removed ($t \cdot ha^{-1}$)	Notes
Slovakia, Krčava, Sobrance	Zachar (1970)	10 20 40 80	2°16′ 2°55′ 4°34′ 4°36′	1.2 8.0 28.4 45.9	measurement on loessic loams in spring 1958, rough tillage
Slovakia, Radvaň, B. Bystrica	Zachar (1970)	10 20 40 80	11°48′ 11°47′ 8°53′ 6°13′	1.7 6.0 18.3 24.5	measurement on loamy soil in spring 1958
Slovakia, Hriňovská	Zachar (1970)	5 10 20 40	20° 20° 21° 20°	130.4 278.3 448.0 630.2	denuded loamy sand on the bank of a dam lake, in 1966
Slovakia, Rakytovce	Kozlík et al. (1961)	25 50 100 200 350	5° 5° 8° 8° 8°	7.5 5.0 20.0 78.0 148.0	soil removed by flood rain – potatoes
USA, Wisconsin, Clinton	Bennett (1939)	11.0 22.1 44.3	16% 16% 16%	159.0 248.0 286.0	sandy loam, stable monitoring in 1933–1936, maize

the weight of the erosion-factor effect f_i. The value of the weight of the effect of a particular erosion factor, i.e., the slope gradient – S, soil erodibility – K, vegetation cover – C and erosion-control provisions – P (see Section 4.3), is a function of its relative distance from the watershed divide – d_r (Fig. 34).

Examples of three slope shapes (average gradient $I = 9\ \%$) are shown in Fig. 35:

- straight slope (denoted *a*)
- convex slope (*o*)
- concave slope (*c*)

If the relative intensity of water erosion on the straight slope is assigned the value $S_p = 100\ \%$, then its value on the convex slope will be $S_p = 120.7\ \%$ and that on the concave slope $S_p = 91.3\ \%$.

As to the influence of topographic conditions in the region on the wind erosion of the soil, particular importance is attached to the effect of the slope's exposure to the most frequent wind, the effect of slope shape (relief of the region) and the effect of slope length.

Table 23

Values of critical gradient for farmed land (ANTAL, 1985)

Soil resistance	Author*						
	1	2	3	4	5	6	7
	Critical gradient (°)						
Poor (loess)	1.5	1–2	5	3.2	3		2
Medium (sandy soil)		3–5		4.5		2–3	5
Good		6–7					7

* 1 – SPIRHANZL (in CABLÍK and JŮVA, 1963),
2 – ZACHAR (in HOLÝ, 1978),
3 – STREMME, MAŘAN and LHOTA (in HOLÝ, 1978),
4 – SVP (in HOLÝ, 1978)
5 – ZACHAR (1970),
6 – SOBOLEV (in HOLÝ, 1978),
7 – KURON (in HOLÝ, 1978)

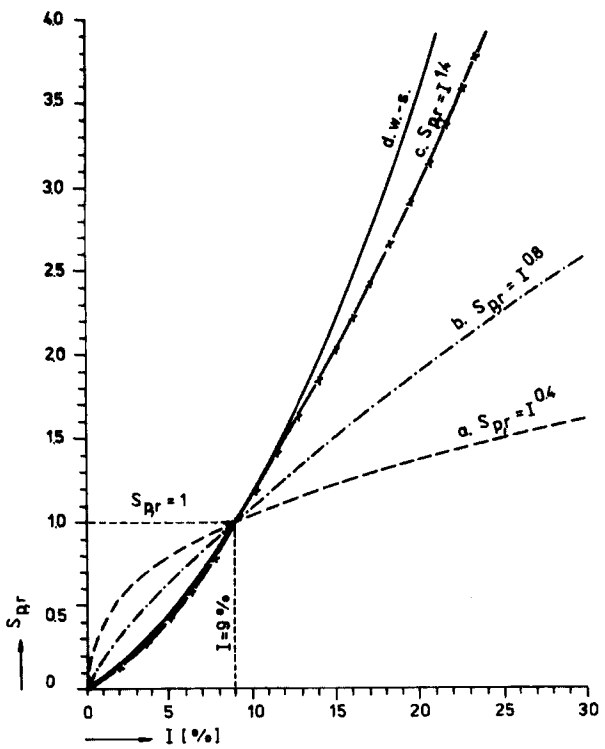


Fig.33 Experimentally determined effect of slope gradient on the intensity of the water erosion of a soil.

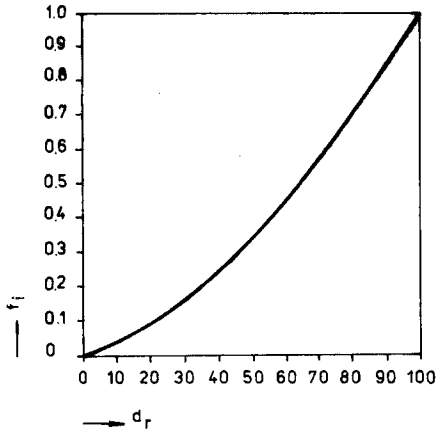


Fig.34 Weight of the f_i of the effect of erosion factors K, S, C and P as a function of their relative distance d_r from the watershed contour line.

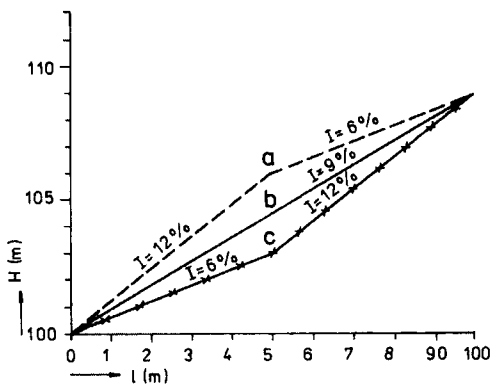


Fig.35 Effect of slope shape on the action of erosion factors K, S, C and P:
a – broken furrowing, b – strip cropping (b_1 – six 40 m strips, b_2 – four 30 m strips, b_3 – root crops, four 20 m strips, b_4 – perennial fodder crops, two 20 m strips), c – strip cropping (c_1 – six 40 m strips, c_2 – four 30 m strips, c_3 – root crops, four 20 m strips, c_4 – winter cereals, two 20 m strips), d – winter cereals, two 20 m strips.

3.2.4.2 The effect of vegetation on soil erosion

Several approaches are available to choose from when evaluating the effect of different types of plant cover of the soil on the intensity of water and wind erosion. The generally applicable approach is to use the classification of vegetation cover types based on their soil-protection effectiveness which is expressed as the relative (percent) intensity of water erosion under each of the types in relation to that for black fallow, i.e., on bare soil:

1. black fallow . . . . . $S_p = 100\ \%$
2. root crops . . . . . $S_p = 60\ \%$
3. spring-sown cereals . . . . . $S_p = 40\ \%$

4. autumn-sown cereals . . . . . $S_p =$ 30 %
5. annual fodder crops on arable land . . . . . $S_p =$ 5 %
6. permanent grassland . . . . . $S_p =$ 1 %
7. woodland . . . . . $S_p <$ 1 %.

Since the soil-protection effectiveness of the vegetation cover of the soil (interception capacity, absorption of kinetic energy, etc. – see Section 3.1.6) also depends on the developmental stage of the crops (except permanent grassland, orchards and woodland), the year can be broken down into five periods, based on changes in the soil-protection effectivity of the crops grown:

Period I – starts by tillage (stubble breaking) and extends until the start of pre-sowing preparation of the soil ("coarse furrow period");

Period II – starts with soil preparation for sowing and ends one month after sowing ("sowing period");

Period III – generally covers the second month after sowing date; with autumn-sown cereals it lasts from the end of the first post-sowing month until the end of April the following year ("stand establishment period");

Period IV – continues from the end of period III until the harvest of the crop ("growth and ripeness period");

Period V – lasts from the harvest of the crop until tillage or until the date of sowing of the subsequent crop if it is sown into the stubble ("stubble period" or "post-harvest residues period").

The contributions of different crops and different cultural practices to the reduction of the water erosion of the soil, compared with black fallow (S_p = 100 % or = 1.0), in the different periods of the year are shown in Tab. 24.

The data in this table can be used for computing the intensity of the water erosion of the soil (Section 4) but the condition is that the distribution of the effect of the rain-erosivity factor R during the year is also known (see Fig. 36). The calculation can be performed according to the formula

$$C_x = c_{1,x}r_1 + c_{2,x}r_2 + \ldots + c_{5,x}r_5, \tag{3.52}$$

where C_x – (= $S_{p,x}/S_{p,o}$) the resultant value of the so-called factor of vegetation cover and cultural practices when the x^{th} crop is grown,

$S_{p,x}$ – soil erosion when the x^{th} crop is grown in the plot in question ($t \cdot ha^{-1} \cdot year^{-1}$),

$S_{p,o}$ – soil erosion when the plot has no vegetation cover (black fallow) ($t \cdot ha^{-1} \cdot year^{-1}$),

$c_{n,x}$ – value of the vegetation cover factor of the x^{th} crop during the n^{th} period of the year (can be found in Tab. 24),

r_n – proportion of the effect of factor R in the n^{th} period of the year (can be found in Fig. 32).

If the distribution of factor R during the year is not known or if the value of factor C is to be determined for permanent grassland, the C factor data for the whole year should be taken from Tab. 25.

Table 24

Values of the factor of vegetation cover and cultural practices "C"

Crop inclusion in crop rotation	Type of crop rotation used	Values of the factor of vegetation cover and cultural practices according to growing periods					
		1	2	3	4	5s	5p
Cereals after 1 year:							
after clovers	OP	0.50	0.55	0.30	0.05	0.20	0.04
	ST	0.02	0.02	0.02	0.02	0.02	0.02
after cereals	OP	0.65	0.70	0.45	0.08	0.25	0.04
	ST	0.25	0.25	0.20	0.08	0.25	0.04
after root crops	OP	0.70	0.75	0.50	0.08	0.25	0.04
	ST	0.70	0.70	0.45	0.08	0.25	0.04
Maize – straw from the preceding crop harvested	OP	0.70	0.90	0.70	0.35	0.70	0.40
		O K	O K	O K			
	ST	0.25–0.7	0.25–0.7	0.2–0.55	0.25	0.60	0.30
Maize – straw from the preceding crop not harvested	OP	0.60	0.75	0.55	0.25	0.60	0.30
		O K	O K	O K	O K	O K	O K
	ST	0.04–0.3	0.04–0.25	0.04–0.2	0.05–0.2	0.25–0.04	0.15–0.3
sown into sod killed by herbicides	perennial fodder crops	0.02	0.02	0.03	0.03	0.05	0.03
	ryegrass as autumn-sown intercrop	0.05	0.05	0.05	0.05	0.15	0.10
Potatoes, sugar beet	in straight rows in any direction	0.65	0.80	0.65	0.30	0.70	
Lucerne						0.02	
Red clover						0.015	
Perennial grassland meadows						0.005	

Explanatory notes: 5s – straw harvested
5p – straw left
O – after cereal
K – after maize
OP – sowing in ploughed soil
ST – sowing into stubble

If there are strips of different types of vegetation cover on different levels on the slope, it is necessary also to take into account the effect of the erosion factor $f_i = f(d_r)$ when specifying the resultant value of factor C.

3.2.4.3 The effect of economic and technical conditions on soil erosion

The effect of the economic and technical conditions on soil erosion manifests itself most intensely after implementing the erosion-control measures. The effects

	I.	II.	III.	IV.	V.	VI.	VII.	VIII.	IX.	X.	XI.	XII.
1st YEAR	CROP	CLOVER							WINTER WHEAT			
	PERIOD	-							1	2	3	
	C	0.015							0.50	0.55	0.30	

2nd YEAR	CROP	WINTER WHEAT							-			
	PERIOD	3			4			5	1			
	C	0.30			0.05			0.2	0.65			

3rd YEAR	CROP	SUGAR BEET							-			
	PERIOD	1		2		3	4	5	1			
	C	0.65		0.60		0.65	0.30	0.70	0.70			

4th YEAR	CROP	UNDERSOWN SPRING BARLEY							CLOVER			
	PERIOD	1		2		3	4		-			
	C	0.70		0.75		0.50	0.08		0.015			

Fig.36 An example of the calculation of the vegetation-cover factor for the whole crop rotation.

of soil use, crops grown, cultural practices, crop rotations and other such factors are not taken into account here because they are comprised within, e.g., the hydrological, topographic and vegetation factors of erosion; hence, the only effect to be considered here is that of the erosion-control (anti-erosion) measures.

The most frequent erosion-control measures include:

- contour cultivation of the soil — P_v
- strip cropping — P_p
- check damming — P_h
- terracing — P_t

The effectiveness of the different erosion-control measures — being a function of (among other factors) the inclination of the plot concerned — is expressed by the value of what is called erosion-control measures factor P_x. The P_x is defined as

$$P_x = \frac{S_{p,o}}{S_{p,x}}, \tag{3.53}$$

where P_x — value of the factor of the x^{th} erosion-control measure,

$S_{p,o}$ — erosion of the soil in a plot ploughed along the slope line ($t \cdot ha^{-1} \cdot year^{-1}$),

$S_{p,x}$ — erosion of the soil in the plot subjected to the x^{th} erosion-control measure ($t \cdot ha^{-1} \cdot year^{-1}$).

The values of factor P_x can be taken either from the data in Tab. 26 or from Fig. 37.

If more than one erosion-control measure is used on different levels of the same slope, the effect of the erosion factor $f_i = f(d_r)$ must also be taken into account.

Table 25

Average annual values of factor $C_{r,i}$ for field crops (TS, 1984)

Field crop	Inclusion in crop rotation		Factor $C_{r,i}$ at sowing		
			sowing into ploughed soil	with any cultural practices	sowing into stubble
Winter wheat	In the first year after clover after root crop, cereal		0.11	0.12	0.02
Rye, winter barley	In the first year after clover after root crop, cereal		0.17	0.17	0.02
Oats, spring wheat	In the first year after clover after root crop after cereal		0.08 0.10	0.10	0.02 0.08
Spring barley	In the first year after clover after root crop after cereal		0.13 0.15	0.15	0.02 0.13
Rape	In the first year after clover after root crop after cereal		0.17 0.24 0.22	0.12	0.02 0.22 0.11
Horse bean (grown for the beans)				0.30	
Silage maize	straw from preceding crop harvested	varying positions in rotation after cereal after maize	0.72		 0.23 0.49
	grass killed by herbicides	after perennial fodder crop after ryegrass		0.03 0.05	
Grain maize	straw from preceding crop harvested	varying positions in rotation after cereal after maize	0.61		 0.25 0.24
	straw from preceding crop left in field	varying positions in rotation after cereal after maize	0.55		 0.05 0.24
	grass killed by herbicides	after perennial fodder crop after reygrass		0.03 0.05	
Early potatoes Medium-early potatoes Late potatoes				0.60 0.45 0.44	
Sugar beet, fodder beet Lucerne Red clover Perennial grassland, meadows				0.44 0.02 0.015 0.005	

Table 26

Values of factor P_x

Anti-erosion measure	Gradient of slope (%) 2–7	7–12	12–18	18–24
Straight rows, any direction	1.0	1.0	1.0	1.0
Contour cultivation,	0.6	0.7	0.9	1.0
at strip field width of	120 m	60 m	40 m	– *)
Strip cropping at a maximum number of strips of:	6 strips	4 strips	4 strips	2 strips
and maximum strip width of:	40 m	30 m	20 m	20 m*)
– alternation of root crops and perennial fodder crops	0.30	0.35	0.40	0.45
– alternation of root crops and winter cereals	0.50	0.60	0.75	0.90
Check damming (broken furrowing along contour lines), pit digging	0.25	0.30	0.40	0.45
Terracing (depending on type)			0.05–0.15	0.05–0.20

Note: *) On longer slopes the field has to be divided according to the given values of admissible strip widths (admissible slope length), otherwise the P value is not applicable

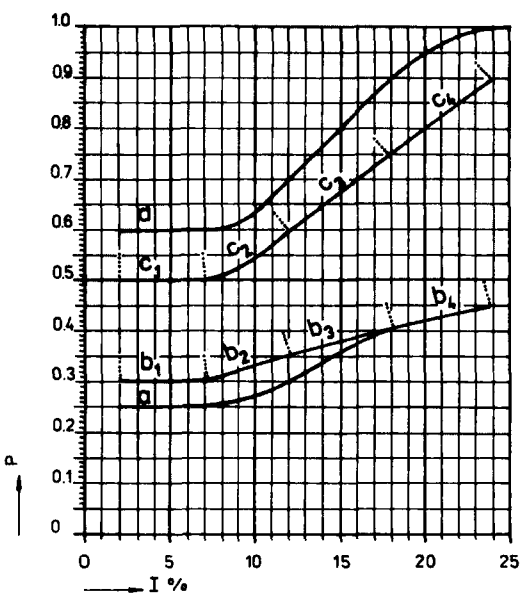


Fig.37 Effect of gradient of slope on the values of the erosion-control measures factor. (After ALENA, 1986.)

CHAPTER 4

Analysis of Erosion

By evaluating the effect of erosion factors on the erosion processes, the removal and transport of eroded material by exogenous forces can be quantified. Both theoretical and empirical methods are used to specify each factor's contribution and describe the course of the erosion processes. The problem studied is very complex and different methods afford different levels of accuracy. The methods used include, in essence, the mathematic expression of the erosion process, the application of empirical formulae and the simulation of erosion.

4.1 Regularities of water erosion

Water erosion takes place during surface runoff, including both the runoff down the soil surface and runoff concentrated in a river network.

Areal surface runoff starts as a coherent layer of water moving down the soil surface. Exposed to the unevenness of the soil surface, this layer breaks into separate runoff lines at a certain distance from the place where the runoff started.

In analysing the areal runoff, two assumptions are used: first, the layer of water running down the surface is coherent and second, its thickness increases with the distance from the start of the runoff (Fig. 38).

The amount of soil removed by runoff and/or the size of the transported soil particles can be determined using these parameters' dependence on the speed of water running down the surface or on the tangential stress.

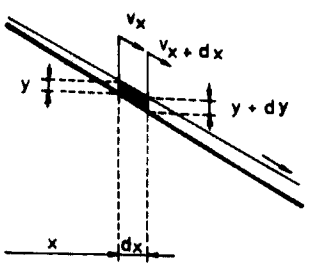


Fig.38 Diagram of overland surface runoff.

4.1.1 Velocity of areal runoff

KOSTYAKOV (1932) and later CABLÍK (1949) put forward an expression for the velocity at which water runs down the soil surface under the given conditions. They used the *Chézy equation:*

$$v_x = c\sqrt{RI}, \tag{4.1}$$

where v_x – velocity of water running down the surface at distance x from the start of runoff ($m \cdot s^{-1}$),
c – the velocity coefficient,
R – hydraulic radius (m),
I – gradient of the slope.

In a very thin runoff layer (thickness y, width 1 m), the hydraulic radius can be described as

$$R = \frac{P}{O} = \frac{1y}{1 + 2y} \doteq y, \tag{4.2}$$

where P – the flow cross-section in the given runoff strip: $P = 1y$,
O – wetted circumference: $O = 1 + 2y$; the value of $2y$ is very low with respect to the width of 1 m, so it can be left out.

The above-mentioned authors recommend that *Bazin's formula* should be used for expressing the velocity coefficient:

$$c = \frac{87\sqrt{R}}{\gamma + \sqrt{R}} = \frac{87\sqrt{y}}{\gamma + \sqrt{y}} \doteq \frac{87}{\gamma}\sqrt{y} = m\sqrt{y}, \tag{4.3}$$

where γ – coarseness coefficient whose values, computed by CHERKASSOV (1950), are shown in Table 27: the data in the table show that a very low value of $\sqrt{y}$ in the denominator can be left out.

m – $\frac{87}{\gamma}$.

After re-formulating expressions (4.2) and (4.3), equation (4.1) will take the form

$$v_x = m\sqrt{y}\sqrt{yI} = m\sqrt{I}y = My, \tag{4.4}$$

Table 27

Values of the roughness coefficient γ (after CHERKASSOV, 1950)

Nature of soil surface	Coefficient γ	$m' = \frac{87}{\gamma}$
Fields ploughed along slope	2.0	43.50
Ploughed fields, smoothed surface	3.5	24.85
Fields overgrown with reeds	4.0	21.75
Fields overgrown with moss	5.0– 6.0	17.40–14.50
Meadows overgrown with short grass	6.0– 8.0	14.50–10.88
Rugged surface with numerous molehills	8.0–15.0	10.88– 5.80

where M – for a constant gradient of slope I: $M = m\sqrt{I}$.

Expression (4.4) in its rearranged form shows the dependence of velocity v_x in profile I (at distance x from the start of runoff) on water depth (thickness of water layer) y. In profile II at a distance of $x + dx$,

$$v_{x+dx} = M(y + dy). \tag{4.5}$$

The amount flowing in profile I is Q_1 and that flowing in profile II is Q_2. The runoff increment dQ is the difference between Q_1 and Q_2:

$$dQ = Q_2 - Q_1 = (y + dy)\, v_{x+dx} - yv_x, \tag{4.6}$$

where $Q_1 = yv_x$,
$Q_2 = (y + dy)\, v_{x+dx}$.

By substituting expressions (4.4) and (4.5) into equation (4.6), we obtain:

$$dQ = (y + dy)M(y + dy) - yMy = 2Mydy + M(dy)^2 \doteq 2Mydy, \tag{4.7}$$

if the very small value for $M(dy)^2$ is left out.

On a strip of a unit width and of a length of dx a runoff increment dQ per unit of time will be brought about by a rain of intensity i:

$$dQ = \varphi i dx, \tag{4.8}$$

where φ – runoff coefficient.

It follows from formulae (4.7) and (4.8) that

$$2Mydy = \varphi i dx \tag{4.9}$$

and after integration:

$$My^2 = \varphi i x$$

and after rearrangement:

$$y = \sqrt{\frac{\varphi i x}{M}}. \tag{4.10}$$

Having introduced (4.10) in the equation for velocity (4.4), the latter equation will take the form

$$v_x = \sqrt{M\varphi i x} = \sqrt{m\varphi i x \sqrt{I}} \qquad (\mathrm{m \cdot s^{-1}}) \tag{4.11}$$

DÝROVÁ (1974), who had objections to using the Bazin formula for the calculation of the velocity coefficient in the Chézy equation, recommends using for the given purpose the *Manning formula:*

$$c = \frac{1}{n} R^{1/6},$$

where n – the coarseness coefficient.
Having introduced the Manning formula in equation (4.1), DÝROVÁ derived the expression for velocity in profile I:

$$v_x = \left(\frac{\varphi i x I^{3/4}}{n^{3/2}}\right)^{2/5} \qquad (\mathrm{m \cdot s^{-1}}). \tag{4.12}$$

4.1.2 Tangential stress with areal runoff

To express tangential stress CABLÍK (1949) used the formula:

$$\tau_x = \gamma y I \qquad (\mathrm{N \cdot m^{-2}}), \tag{4.13}$$

where τ_x – tangential stress in profile I,
γ – specific gravity of water ($\mathrm{N \cdot m^{-3}}$),
y – depth of water in profile I,
I – gradient of slope.
Introducing the derived relationship for y (4.10) in this equation, we obtain:

$$\tau_x = \gamma I \sqrt{\frac{\varphi i x}{M}} = \gamma I \sqrt{\varphi i \frac{x}{m\sqrt{I}}} \qquad (\mathrm{N \cdot m^{-2}}). \tag{4.14}$$

As in the preceding case, DÝROVÁ (1974) used the Manning form of the velocity coefficient to obtain:

$$\tau_x = \gamma I^{7/10} (x \varphi i n)^{3/5} \qquad (\mathrm{N \cdot m^{-2}}). \tag{4.15}$$

4.1.3 Admissible slope length

The erosion processes can be studied on the basis of the dependence of velocity or tangential stress on the distance from the origin of runoff. The distance at which the velocity of running water or the tangential stress are high enough to move grains of a certain size (i.e., a certain erosion phenomenon takes place) is referred to as the limit length of slope (l). If at this distance the areal runoff changes into concentrated runoff, HOLÝ (1978) refers to it as the critical length of slope.

Depending on the local conditions and on the requirements of farming practice, conditions can be determined for characterising the admissible slope length which is usually longer than the critical slope length if there is any difference at all. The calculation of the admissible slope length is based on the assumption that in a plot whose width is less than or equal to the admissible slope length, there are no erosion processes that would be harmful to crop growing.

Hence, for $v_x = v_k$ and $x = l_v$, equation (4.11) can be used to compute the dependence of the critical slope length l_v on the limit velocity v_k (CABLÍK, 1949):

$$v_k = \sqrt{m \varphi i l_v \sqrt{I}},$$

$$l_v = \frac{v_k^2}{m\varphi i\sqrt{I}} \quad \text{(m)}. \tag{4.16}$$

Similarly, for $\tau_x = \tau_k$ and xl_τ, equation (4.14) will allow of determining the dependence of the critical slope length l_τ on the limit tangential stress τ_k:

$$\tau_k = \gamma I \sqrt{\frac{\varphi i l_\tau}{m\sqrt{I}}},$$

$$l_\tau = \frac{\tau_k^2 m}{\gamma^2 I^{3/2}\varphi i} \quad \text{(m)}. \tag{4.17}$$

If corresponding values of the parameters are used, equations (4.16) and (4.17) will give the same values.

Using the Manning formula for the velocity coefficient, DÝROVÁ suggests the following expressions for the critical slope length:

$$l_v = \frac{n^{3/2} v_k^{5/2}}{\varphi i I^{3/4}}, \tag{4.18}$$

$$l_\tau = \frac{\tau_k^{5/3}}{\gamma^{5/3}\varphi i n I^{7/6}}. \tag{4.19}$$

As in the preceding case, equations (4.18) and (4.19) are supposed to give the same values.

However, CABLÍK recorded large differences, so he recommended using formula (4.17) because in the majority of cases (4.16) gave higher values of slope length: this was so because the ideal runoff in a coherent layer soon changes into concentrated runoff and the actual velocity is thus higher than the velocity theoretically derived in formula (4.11).

Taking this fact into account, KOSTYAKOV (1951) introduced a correcting coefficient in equation (4.11):

$$v_x = \nu\sqrt{m\varphi i x\sqrt{I}}, \tag{4.20}$$

where: ν – correction coefficient depending on the gradient of slope: it increases with increasing slope gradients $\nu \geqq 1$.

VESELÝ (1961) derived the value of this coefficient: $1 \leqq \nu \leqq 3$.

A similar solution is recommended by MIROKHULAVA (1970). Failure to take into account the reason for which the correction coefficient is introduced led to the above-mentioned discrepancy between the otherwise mutually coresponding formulae (4.16) and (4.17).

For using the derived expressions, these parameters were derived or determined for formulae (4.16–4.19).

CHERKASSOV (1950) gives the values for γ the coarseness coefficient. These values are shown in Tab. 27 with converted values of m. The value for a

ploughed field with a smoothed surface is most frequently used: $m = 24.85$.

DÝROVÁ (1974) recommends using the value of 0.03 for the coarseness coefficient n.

The runoff coefficient φ is, as a rule, chosen from different authors' guide values. The initial KOSTYAKOV (1971) data are shown in Tab. 28. Based on measurements in runoff areas, reliable values for different gradients of slope were derived from the infiltration – surface runoff relationships by DVOŘÁK (1975) (Tab. 29). Multiple runoff coefficients after HÁRTL (HOLÝ, 1978) can be used with advantage:

$$\varphi = n_1 n_2 n_3 n_4,$$

where: φ – runoff coefficient,
n_1 – coefficient showing the effect of the length of the valley,
n_2 – coefficient showing the effect of afforestation of the terrain,
n_3 – coefficient showing the effect of inclination of the terrain,
n_4 – coefficient showing the effect of soil permeability.

The data are shown in Tab. 30. For simplification, the specific gravity of water is considered without the load of suspended sediment (i.e. 9810 $N \cdot m^{-3}$).

For the admissible speed, CABLÍK used the formula derived by VELIKANOV (1948):

$$v_{adm} = 3.14 \sqrt{15d + 0.006},$$

Table 28

Values of the runoff coefficient φ (after KOSTYAKOV, 1951)

Soils in the drainage area	Gradients of the drainage area		
	low < 0.01	medium 0.01 – 0.05	high > 0.05
Highly permeable	0.10 – 0.20	0.15 – 0.25	0.20 – 0.30
Medium permeable	0.15 – 0.25	0.20 – 0.30	0.25 – 0.40
Fairly permeable	0.20 – 0.30	0.25 – 0.45	0.35 – 0.60
Poorly permeable	0.25 – 0.40	0.30 – 0.60	0.50 – 0.75
Frozen	0.35 – 0.60	0.40 – 0.75	0.80 – 0.95

Table 29

Values of the runoff coefficient φ (after DVOŘÁK, 1975)

Soils \ I %	4	6	8	10	12	14	16	18	20	26	30
Medium permeable	0.25	28	32	35	38	40	42	44	46	50	0.52
Fairly permeable	0.35	45	49	51	58	60	62	64	66	70	0.73
Poorly permeable	0.45	51	60	67	71	75	79	81	84	86	0.88

Table 30

Characteristics of the runoff coefficient (after HÁRTEL, 1938)

With a valley length of	0.2	0.3	0.4		up to 10 km
n_1	0.9	0.85	0.8		0.55
With an afforestation coverage	100	75	50	25	0%
n_2	0.6	0.7	0.8	0.9	0.95
If the relief is:	very steep		mountainous		hilly
n_3	0.90		0.85		0.80
If the soil is:	impervious	poorly permeable	medium permeable		highly permeable
n_4	0.90	0.80	0.75		0.70

where v_{adm} – extreme velocity at which water runs down the surface ($m \cdot s^{-1}$),
d – mean grain diameter (m).

For $d = 0$ the v_k is $0.24\ m \cdot s^{-1}$ and this value was recommended by CABLÍK (1949) for use in the above-mentioned formulae. HOLÝ (1978) gives values derived for different types of soil (Tab. 31). DÝROVÁ (1974) recommends the values given in Tab. 32. The issues of the determination of admissible velocity have been studied by a number of investigators (HOLÝ, 1978).

The data used so far to express the admissible shear stress are the derived values given by STREL and later by CHERKASSOV ($\tau_k = 10.79\ N \cdot m^{-2}$), corresponding to loamy soil. DÝROVÁ (1974) recommends the values shown in Tab. 32.

Characteristics of design flood rain depend on the local conditions. CABLÍK (1963) recommends an intensity of 97 litres per second per ha and a duration of 45–60 min. Cablík's data on the dependence of admissible slope length on

Table 31

Values of v_{adm} for different types of soils

Content of category *I* particles (%)	Soil	D_{50} (mm)	v_{adm} ($m \cdot s^{-1}$)
0–10	sandy earth	0.55	0.37
10–20	loamy sand	0.20	0.31
20–30	sandy loam	0.12	0.28
30–45	loamy earth	0.03	0.26
45–60	clay loam	0.01	0.25
60–75	clayey earth	0.002	0.24
>75	clay, or clayey earth readily breaking down in water	0.001	0.24

Table 32

Values of v_k and τ_k (after DÝROVÁ, 1971)

Soils	v_k	τ_k
Sandy loams	0.24	2.5
Loams, clay loams	0.3	3.8
Loamy sands	0.4	6.6
Clays	0.5	10.9
Structure soils (loamy sands)	0.7	19.5

different gradient at $\varphi = 1$ are shown in Fig. 39. However, the time or the duration of design rain depends on the time of concentration which, in turn, depends on the inclination of the terrain. The most commonly used time (duration) of design rain is 15 – 30 min, the frequency of occurrence being 0.5 – 0.2. The problem of choosing the time of duration for $t > 15$ min is eliminated by introducing the LINDLEY formula for the dependence of rain intensity on rain duration

$$i = \frac{C}{t^a} \tag{4.23}$$

in the areal runoff equations. The following equations are thus obtained (DVOŘÁK, 1969):

$$l_v = A\varphi^{-1/\beta}\,\frac{1}{I^{2\beta}}, \tag{4.24}$$

where $A = \frac{v_k}{2}\left(\frac{2v_k}{mC}\right)^{1/3}$;

$$l_\tau = Bo^{\beta-\frac{1}{\beta}} I^{-\frac{2+\beta}{2\beta}}, \tag{4.25}$$

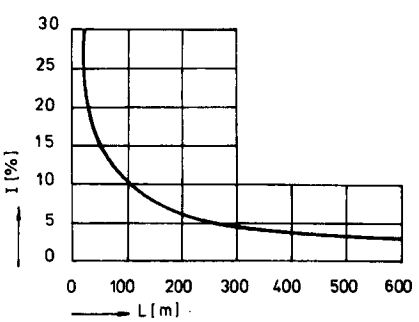


Fig.39 Dependence of the admissible slope length on gradient. After CABLÍK (1963): I – slope gradient, L – admissible slope length = field width.

where $B = \frac{m\tau_k}{2\gamma}\left(\frac{2\tau_k}{C\gamma}\right)^{1/\beta}$

$\beta = 1 - \alpha$

C, α – parameters of equation (4.23).

Methods for the determination of runoff using the constant runoff coefficient φ in equations (4.16), (4.17), (4.18), (4.19), (4.24) and (4.25) are somewhat inaccurate; this is resolved by using the method of determining the admissible slope length from the runoff values treated according to HOLÝ (1978):

$$l = am\,\frac{\tau_k^2 B}{\gamma(aA\gamma I - \tau_k)}\,I^{-1/2}, \tag{4.26}$$

where A and B – characteristics of the dependence of q on t,

a – coefficient depending on the soil conditions.

4.2 Regularities of wind erosion

Strong winds on large areas, especially ploughed fields, intensify erosion processes. Wind erosion has the following three stages:

– destruction of soil surface by the mechanical force of the wind (abrasion),

– transport of the eroded earth particles (deflation),

– sedimentation of the transported earth particles in another place when the force of the wind declines (accumulation).

Wind erosion is a complex process, depending mainly on the strength of the wind and on the condition of the soil surface. The wind, on the one hand, removes fine soil particles, fertilizers and seeds, exposes plant roots and kills young plants; on the other hand, it accumulates the transported material on roads, covers the vegetation etc. Wind erosion most frequently affects bare soil without vegetation. Vegetation and plant residues check the windspeed on the soil surface and the plant roots bind and reinforce the soil.

The soil particles driven or carried by wind show three types of movement:

– particles larger than 1000 μm are rolled on the soil surface,

– particles 50 to 1000 μm in size jump (saltate) on the surface,

– particles smaller than 50 μm are seized and carried by wind (windborne).

The transport and grading of soil particles by the wind depend, first of all, on wind strength and on the size and resistance of the soil particles. The main factors underlying the resistance of soil particles include the soil texture and moisture, the treatment of the soil surface and the plant cover. A coarse soil surface reduces the surface windspeed by about 40%, compared with a smooth surface.

Soil texture is among the key soil factors on which wind erosion depends. The size of the soil particles is the most important, whereas differences in the shape of the particles have a minor influence. Particle sizes of 0.25 to 0.4 mm form the largest proportion of the eroded material. Small particles (size below 0.06 mm) are not so amply represented: the wind-erosion resistance of the fine dust particles is

due to their greater adherence to the large particles which can resist wind. Particles larger than 0.8 mm are also fairly resistant to wind erosion and can be generally considered as nonerodible (PASÁK, 1965).

Sandy soils are the most susceptible to wind erosion. The larger the proportion of clay particles (sizes below 0.01 mm), the lower the erodibility of the soil. The dependence of the wind erodibility of the soil on the content of clay particles is logarithmic and can be expressed by the equation (PASÁK, 1965):

$$E = 2.28 \cdot 10^2 \cdot 10^{-0.0787M} \qquad (4.27)$$

$$(\log E = 2.3587 - 0.0787M),$$

where E – soil erodibility by wind ($g \cdot m^{-2}$),
M – content of clay particles (<0.01 mm) in the soil (% by weight).

Soil erodibility by wind shows a closer dependence on the content of nonerodible particles (>0.8 mm) in the dry soils, containing a minimum quantity of secondary aggregates. The dependence is logarithmic and is expressed by the equation (PASÁK, 1965):

$$E = 2.105 \cdot 10^2 \cdot 10^{-0.358P} \qquad (4.28)$$

$$(\log E = 2.32328 - 0.0358P),$$

where P – content of nonerodible particles (>0.8 mm) in the soil (% by weight).

The proportion of particles larger than 0.8 mm in the dry soil is considered as the key criterion for the evaluation of soil erodibility by wind.

Due to surface tension, water in the soil enhances the cohesion of the soil particles and the formation of secondary aggregates – lumps or clods or surface crust.

A significant relationship between the erodibility of the soil by wind and soil moisture exists in the sandy soils and loamy sands. The larger the content of clay particles in the soil, the lower the effect of moisture on the removal of soil particles by wind. Hence, in the medium-heavy and heavy-textured soils the influence of water is not so strong as in light-textured soils, particularly those of the sandy class (PASÁK, 1966).

The surface of the soil in a field is not uniform: it comprises aggregates of different sizes and of different capability to form surface crust. Spots with a small proportion of nonerodible aggregates and a low crust-forming capability are foci of wind erosion and sources of abrasive particles which successively break up the stronger soil surface. In this way, the whole field rapidly yields to wind erosion.

The relationship between wind speed and soil erodibility by wind is linear and highly significant. The starting wind speed at which the soil particles begin to move ranges between 2 and 3 $m \cdot s^{-1}$ (measured near soil surface). This, upon conversion, corresponds to a slight to moderate wind (2 to 3 on Beaufort scale – 10 to 14 km per hour). The critical windspeed at which the wind erosion is higher than admissible depends on the type and moisture of the soil (PASÁK, 1967) (Tab. 33).

It follows from data in Tab. 33 that a larger than admissible amount of soil is removed

– in dry light-textured soils (sands and loamy sands) at a wind of 3° Bf,

Table 33

Critical windspeeds in km·h^{-1} for different soil types

Soils	Dry soil	Wet soil
Sands	16	38
Loamy sands	16	82
Sandy loams	32	54
Loams	106	106

- in wet loamy sands at a strong windstorm of 10° BF,
- in dry sandy loams at a high wind of 5° Bf,
- in wet sandy loams at a stormy wind of 8° Bf,
- in dry and wet loamy soils at a very strong windstorm of 11° Bf,

where °Bf – degrees of the Beaufort scale.

The interaction of the main factors of soil erodibility by wind is expressed by the erodibility equation (PASÁK, 1967) (Fig. 40):

$$E = 22.02 - 0.72P - 1.69V + 2.64R, \tag{4.29}$$

where E – erodibility of the soil by wind (g·m^{-2}),
P – content of nonerodible particles in the soil (>0.8 mm) (% by weight),
V – relative soil moisture $V = \frac{V_o}{V_n}$, where V_o is the instantaneous moisture (% by weight), $V_n = \frac{M}{2.4}$,
R – windspeed near the soil surface (m·s^{-1}).

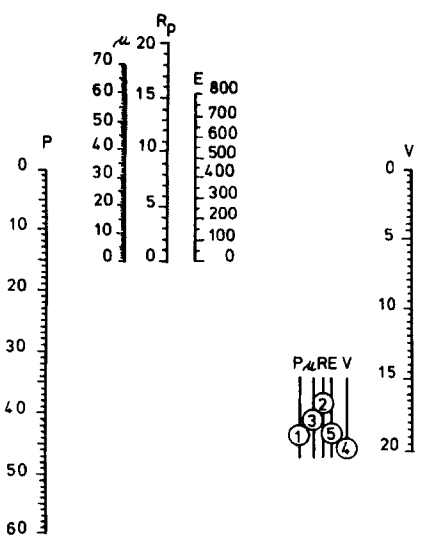


Fig.40 Nomogram for the determination of soil erodibility by wind, E, in kg per ha.

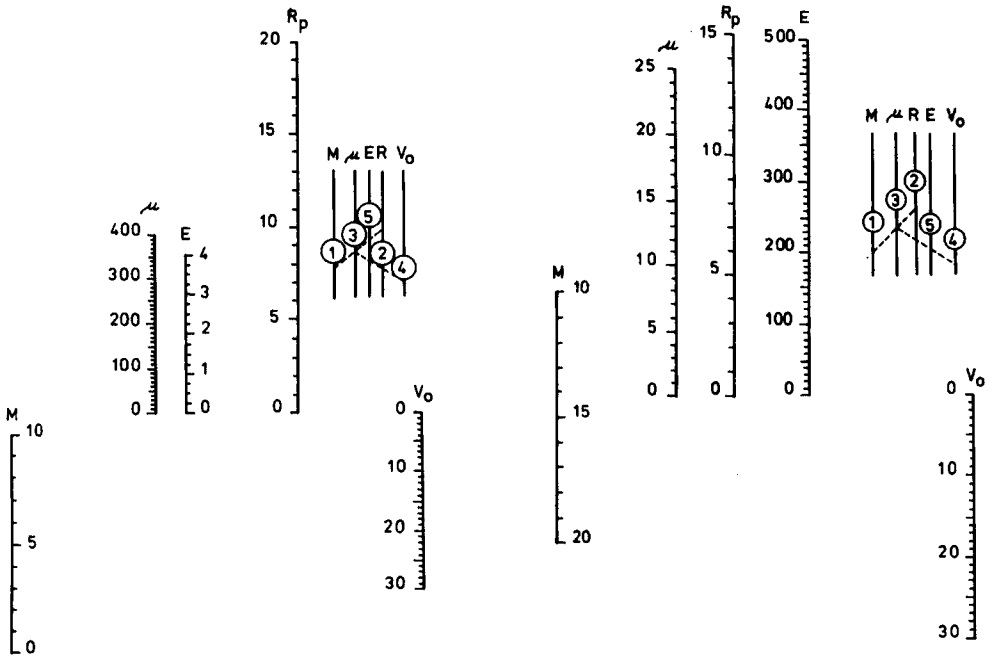
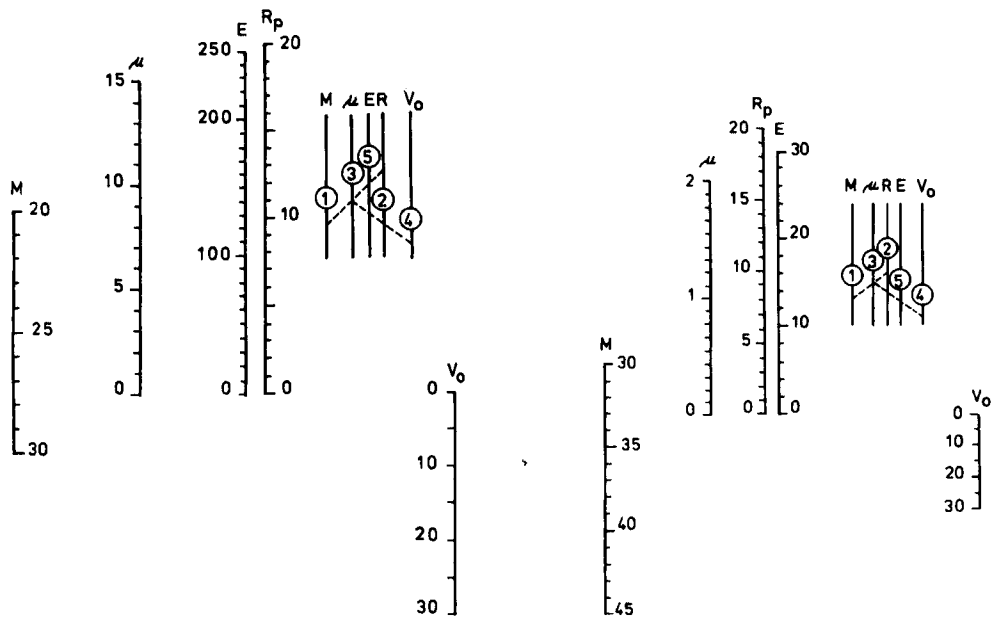


Fig.41 Nomogram for the determination of sandy soil erodibility by wind, *E*, in kg per ha. Δ
Fig.42 Nomogram for the determination of loamy sand erodibility by wind, *E*, in kg per ha. Δ
Fig.43 Nomogram for the determination of sandy loam erodibility by wind, *E*, in kg per ha. ∇
Fig.44 Nomogram for the determination of loamy soil erodibility by wind, *E*, in kg per ha. ∇



It is easier in practice to derive the expression of soil erodibility by wind from its dependence on the content of clay particles and on instantaneous moisture. However, such an expression would be very inaccurate if it applied to the whole range of soils, so it had to be calculated separately for each soil class.

Equation of soil erodibility by wind (Figs. 41−44) for

- sandy soils: $E = -269.24 + 24.5M - 1.65V_o + 20.92R$, (4.30)
- loamy sands: $E = 8.95 - 0.63M - 0.51V_o + 1.22R$, (4.31)
- sandy loams: $E = 16.09 - 0.58M - 0.18V_o + 0.42R$, (4.32)
- loamy soils: $E = 1.55 - 0.05M - 0.01V_o + 0.08R$, (4.33)

where M − content of clay particles (<0.01 mm) in the soil (% by weight),
V_o − instantaneous soil moisture (% by volume).

The values of soil erodibility by wind apply to smooth flat soil surfaces, free of vegetation.

The *effect of vegetation* on wind erosion can be evaluated using the equation derived by BARAEV (1974):

$$E = 10^{(a - bH - cS)}, \tag{4.34}$$

where E − erodibility of soils by wind ($g \cdot m^{-2}$),
H − cloddiness, i.e. content of aggregates larger than 1 mm (%),
S − amount of vegetation (stubble) on soil surface (number of pieces per 1 m^2),
a, b, c − exponents depending on soil type. Their values are within the following ranges: $a = 3.2 - 3.9$; $b = 0.021 - 0.039$; $c = 0.0025 - 0.0052$.

It follows from this equation that a 1% increase in the content of aggregates larger than 1 mm has the same effect on the erodibility of the soil by wind as an increase in the amount of vegetation (stubble) by 8−10 pieces per 1 m^2. This implies that it would be very costly and in fact impossible to provide reliable wind-erosion control only be means of a higher cloddiness, with no vegetation.

To enable an all-round evaluation and solid prediction of wind-erosion processes, CHEPIL and WOODRUFF (1963) put forward their *wind-erosion equation:*

$$G = J'DCf(L')f(S), \tag{4.35}$$

where G − potential loss of soil by wind ($t \cdot ha^{-1} \cdot year^{-1}$),
J' − erodibility factor of a flat and rolling surface,
D − factor of the coarseness of the waves of the microrelief,
L' − factor of field length,
S − factor of equal amount of vegetation cover.

The relationship of the first three factors to the potential loss of soil is simple, but factors L' and S are inherently tied in with other factors. The use of this equation in project design practice and implementation is limited by the inaccessibility of the values of the separate factors.

4.3 Modelling erosion processes

The erosion processes can be generally expressed by a formula determining the relationship between its resultant intensity (i.e., the gravity or volume loss of soil from a unit area for a given period of time) and the erosion factors.

The empirical model of erosion processes can be written in its general form (HOLÝ, 1978):

$$S_P = f(X_K, X_H, X_M, X_P, X_G, X_V, X_T, X_{EK}), \tag{4.36}$$

where S_P – intensity of erosion process (removal of soil),
X_K – climatic factor,
X_H – hydrological factor,
X_M – morphological factor,
X_P – soil factor,
X_G – geological factor,
X_V – vegetation factor,
X_T – economic and technical factor,
X_{EK} – socioeconomic factor.

It follows from the factors described in Chapter 3 that this model is very difficult to solve because the different factors interact and vary in both space and time and because it is costly and laborious to collect so many data. Many attempts have been made to apply relationships with a lower number of parameters to express the intensity of erosion, e.g., by BOGARDI (1974), DEMIRÖS (1973), GRAF (1971), HUDSON (1971), FREVERT (1955), MUSGRAVE (1974) and others, including the Czech authors HOLÝ (1964), ZDRAŽIL (1963), STEHLÍK (1970) and PRETL (1973).

4.3.1 Wischmeier and Smith's empiric model of the erosion processes

WISCHMEIER and SMITH (1965, 1975, 1976) conducted extensive investigations of erosion processes under the natural and farming conditions of the USA and used the results to include the effect of further factors in the FREVERT and MUSGRAVE calculations. The resultant equation, considering the quantitative effect of six key erosion factors, is called the universal equation of soil loss:

$$S_P = RKLSCP. \tag{4.37}$$

The contribution of each term in the equation to erosion intensity was investigated on a so-called unit plot with precisely defined parameters: plot length 22.13 m, gradient 9%, perennial fallow cultivated across the contour lines.

In equation (4.36):

S_P – annual loss of soil ($t \cdot ha^{-1}$) or the so-called admissible soil loss for the given plot,

R – factor of the erosion effect of rain and surface runoff, defined as the product of the kinetic energy of rain and its highest 30-minute intensity;

$$R = Ei_{30}, \tag{4.38}$$

where the kinetic energy of rain can be computed from the following relationship:

$$E = (206 + 87 \log i)\, h, \tag{4.39}$$

where E – kinetic energy of precipitation ($J \cdot m^{-2}$),
i – intensity of precipitation ($cm \cdot h^{-1}$),
h – amount of flood rain (cm),

K – factor of soil erodibility, defined as the removal of soil particles from the unit plot per unit of rain factor R. The erodibility factor is determined either experimentally or mathematically, using its dependence on the following five key characteristics of the soil: content of clay and dust particles (< 0.10 mm), content of sandy particles (0.10 – 2.00 mm), content of organic matter (% of humus), soil texture and soil permeability; a nomogram was used for this purpose (Fig. 45),

L – factor of slope length: it shows the ratio of the loss of soil from the plot investigated to the loss of soil from the unit plot. It can be expressed by the formula:

$$L = \left(\frac{d}{22.13}\right)^{P}, \tag{4.40}$$

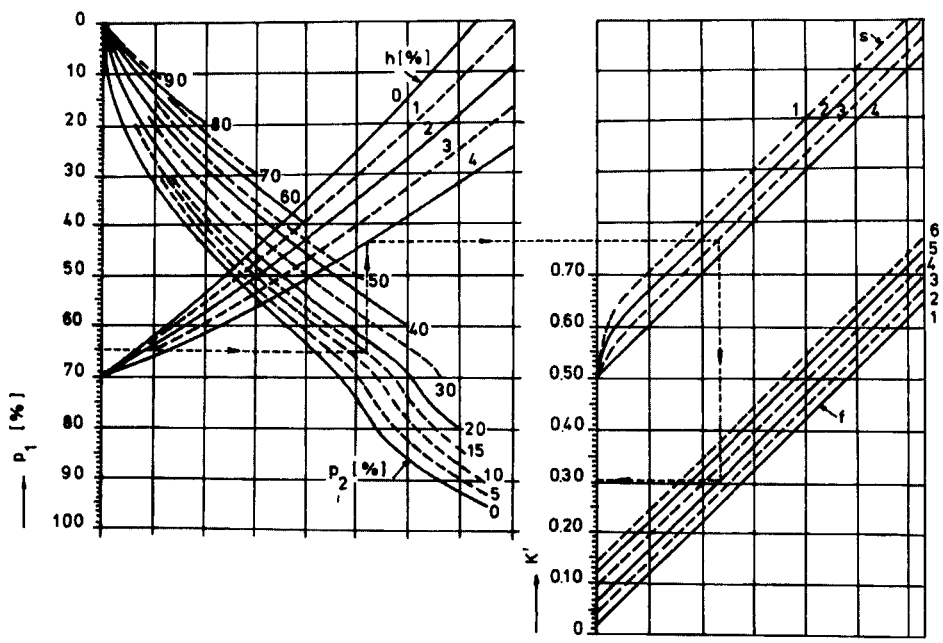


Fig.45 Nomogram for the determination of factor K: K – value of factor K, p_1 – content of soil-particle category 0.001 – 0.1 mm in %, p_2 – content of soil-particle category 0.1 – 2.0 mm in %, h – humus content in %, s – soil structure: 1 – crumb, 2 – crumb, granular, 3 – lumpy, 4 – polyhedric, prismatic, leaflike, platy, fused, f – permeability: 1 – high, 12 $cm \cdot h^{-1}$, 2 – medium to high, 6.0 to 12.0 $cm \cdot h^{-1}$, 3 – medium, 2.0 to 6.0 $cm \cdot h^{-1}$, 4 – low to medium, 0.5 to 2.0 $cm \cdot h^{-1}$, 5 – low, 0.1 to 0.5 $cm \cdot h^{-1}$, 6 – low, 0.1 $cm \cdot h^{-1}$.

where d – unbroken length of slope measured from the watershed divide (m),

P – exponent whose value is 0.3 to 0.6. For slopes with gradients of $I < 10\%$ the p is usually 0.5, for slopes with gradients $I > 10\%$, $P = 0.6$

The values of factor L are given in Tab. 34 and on the graph in Fig. 46,

S – factor of slope gradient: it expresses the ratio of the loss of soil from the plot investigated to the loss of soil from the unit plot which is inclined from the horizontal plane by 9%. It can be computed from the following formula:

$$S = \frac{0.43 + 0.30I + 0.043I^2}{6.613}, \tag{4.41}$$

Table 34

Values of the slope length factor (L)

d (m)	5	10	15	20	30	40	40	60	80	100	150
L	0.48	0.68	0.82	0.95	1.17	1.35	1.52	1.66	1.91	2.13	2.61
d (m)	200	250	300	350	400	450	500	600	700	800	900
L	3.02	3.38	3.69	3.99	4.27	4.52	4.77	5.22	5.62	6.04	6.39
d (m)	1000	1100	1200	1300	1400	1500					
L	6.75	7.07	7.39	7.69	7.98	8.26					

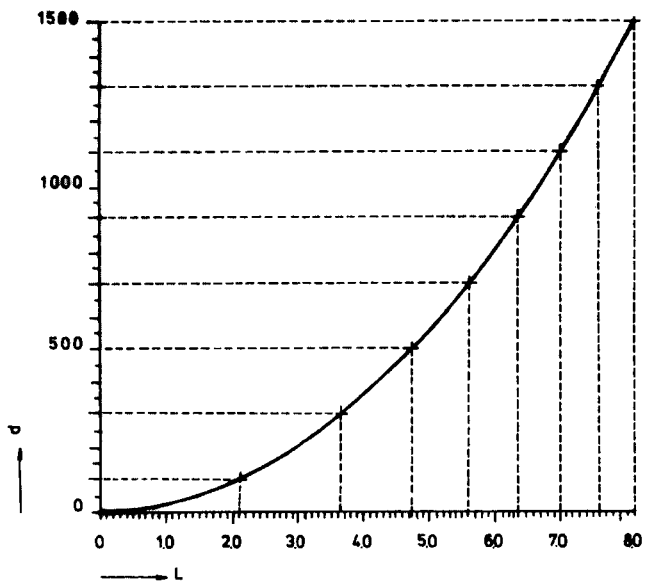


Fig.46 The values of the slope length factor L depending on length d in m.

where: I – slope gradient (%).
The data on factor S are given in Tab. 35 and on the graph in Fig. 47. The authors of the universal equation of soil loss recommend using a combined (topographic) factor of slope length and gradient, expressed by the formula:

$$LS = \frac{d^P}{100}\,(1.36 + 0.97I + 0.1385I^2), \qquad (4.42)$$

where d – length of slope measured from the watershed divide (m),
I – gradient of the slope (%),
Factors LS can be determined from the graph in Fig. 48,

C – factor of vegetation cover and cultural practices used: it gives the ratio of the loss of soil from the plot investigated to the loss of soil in the unit plot (black fallow), all conditions being otherwise the same. With respect to the change in the protective effect of vegetation and soil cultivation during the year, the authors broke the year down into five periods as described in

Table 35

Values of the slope gradient factor (S)

I (%)	2	3	4	5	6	7	8	9	10	11	12
S	0.18	0.26	0.35	0.45	0.57	0.70	0.84	1.0	1.17	1.35	1.55
I (%)	13	14	15	16	17	18	19	20	21	22	23
S	1.75	1.97	2.21	2.46	2.72	2.99	3.27	3.57	3.89	4.21	4.55
I (%)	24	25	26	27	28	29	30				
S	4.90	5.26	5.64	6.03	6.43	6.85	7.28				

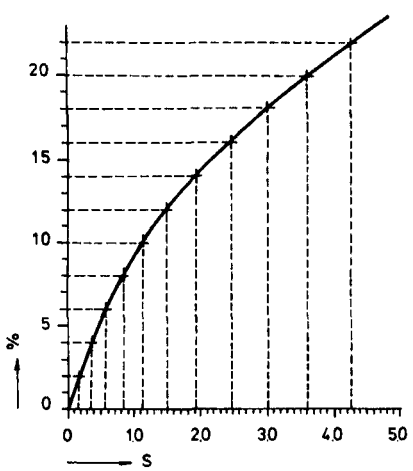


Fig.47 Values of the slope gradient factor S depending on gradient in %.

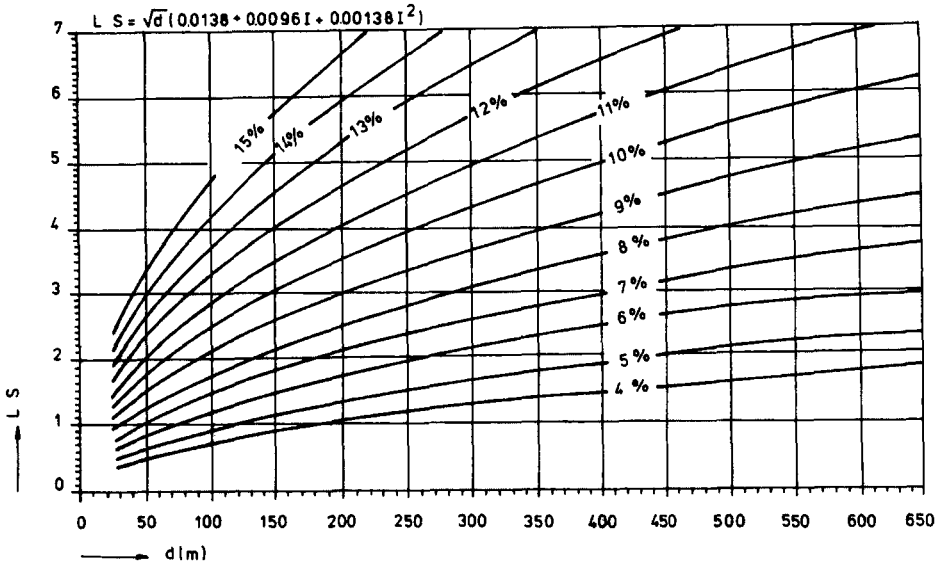


Fig.48 Values of the topographic factor LS depending on slope length in m and gradient in %.

Section 3.2.4.2. For this factor, a table of C values was compiled from a large body of data collected at US experimental stations.
The data on factor C are given in Tab. 25,

P – factor expressing the effect of erosion-control measures: it gives the ratio of the loss of soil from the plot investigated to the loss of soil from the unit plot cultivated across the contour line.
The table of factor P values (Tab. 26) was compiled for different methods and practices of cultivation, including terracing.
This model is suitable for straight slopes. Where the gradient varies, it is recommended to divide the slope into smaller segments of the some gradient and to calculate the factor LS by the equation

$$LS = \frac{\sum_{j=1}^{n} S_j d_j^{1.5} - S_j d_{j-1}^{1.5}}{d/(22.13)^{0.5}}, \tag{4.43}$$

where S_j – factor of the slope gradient of the j^{th} segment, calculated by equation (4.41) for $S = S_j$,
d – unbroken length of slope (m),
d_j – distance from the upper end of the slope to the lower boundary of the j^{th} segment (m),
n – number of segments of the slope.

A similar procedure can be used to also include in the calculation the effect of changes in soil properties, i.e., changes in factor K.

The empiric model of the erosion process described here is based on a vast body of information on erosion phenomena in the USA, gathered during a long period of 50 years. As such, it has a very wide applicability and is used in many countries of the world where it is adjusted to fit the local conditions.

4.3.2 Mathematical models of erosion processes

As in other areas of science and research, scientists are trying to computerise the investigation of erosion processes, using mathematical relationships. Such models involve very complex dynamics of erosion processes based on theoretical relationships of several disciplines of science.

The models usually break the process down into separate stages within which they describe the release of soil particles and their transport by surface runoff. The links between the separate stages are represented by mathematical and empiric relationships.

Of the great number of existing models, the following may be regarded as being of the greatest practical importance:

CREAMS (Chemicals, Runoff and Erosion from Agricultural Management Systems). The model is designed to simulate the hydrological and erosion processes and transport of selected chemicals (N, P, pesticides) in areas with homogeneous soils, consistent management and regular distribution of rainfall. The maximum size of the area that can be treated by the model depends on the homogeneity of the territory, hence, it cannot be greater than a hundred hectares.

SWRRB (Simulator for Water Resources in Rural Basins). This model is designed for the simulation of hydrological processes and transport of washed material in farmed watersheds and for the evaluation of how these processes are affected by changes in the system of management. The watershed is represented by one typical profile of areal surface runoff and one typical profile of concentrated runoff. The maximum size of the watershed that can be treated by the model is 10 000 ha.

ANSWERS (Areal Nonpoint Source Watershed Environment Response Simulation) enables to simulate hydrological processes and the transport of washed material from non-homogeneous watershed up to 10 000 ha in size. The watershed is represented by a system of homogeneous square elements, each acting as a separate hydrological unit. Hydrological and erosion processes take place in such a unit and their results are transferred to the adjacent elements. The model is designed for solutions relating to the watershed response to separate design rainfall levels.

AGNPS (Agricultural NonPoint Source). The model helps to find solutions to problems relating to runoff, erosion and transport of chemicals (N, P) from a separate rainfall in watersheds up to 10 000 ha in size, whose homogeneity is provided by a network of square elements.

SMODERP (Simulation Model of Surface Runoff and Erosion Processes). This model uses physical principles to treat the rainfall – runoff relationships and erosion processes on hillslopes (fields). It simulates surface runoff and erosion processes resulting from rainfalls of varying intensities in areas up to 1 km^2 in size in which the morphological, soil and vegetation relationships are non-homogeneous. The model can be used for the determination of: a) characteristics of surface runoff (runoff volume, surface flow rate, depth, velocity and tangential stress) in

selected profiles of the hillslope under study and in selected intervals of time at the beginning of the rainfall; b) the admissible slope (field) lengths, the determination being based on the maximum non-scouring tangential stress and maximum non-scouring surface runoff velocity; c) losses of soil (transport of soil outside the borders of the hillslope [field]).

WEPP (Water Erosion Prediction Project) is a new technology devised to replace the USLE (Universal Soil Loss Equation) and RUSLE (Revised Universal Soil Equation) for soil loss prediction in the United States. WEPP is planned to produce three versions to predict soil erosion on different scales. These are the profile version, watershed version and grid version. The profile version is to replace the USLE, with the ability to compute deposition that occurs on a hillslope, as well as the amount of material that is eroded. The profile version computes sediment delivery to a channel, and then sediment delivery from multiple fields through channels and impoundment is estimated, using either the watershed or grid version. All versions were to be delivered for public use in late 1993. All versions of the model operate on a daily simulation. They simulate on a daily basis the conditions of fields and their response to daily climate. The profile version is the basic version for simulating upland soil erosion and is a component of both the watershed and grid versions.

More information on the mathematical models is given in DE ROO (1993).

CHAPTER 5

Erosion Control to Protect the Soil

Erosion control is a package of measures aimed at
a) preventing the development of damaging erosion,
b) reducing intensity of erosion to an admissible (tolerable) level or lower.

Damaging erosion is primarily a consequence of a change in the mode of use of the land such as the conversion of forest land into arable land, conversion of permanent grassland into arable land, transition from small-scale farming to large-scale agricultural production and so on, or a consequence of wrong technical interventions such as the construction of an unsuitable road network, waterways, etc. Such changes and interventions break the long-established balance between the effect of erosion factors and the soil's ability to withstand the destructive and transporting activity of such factors.

Any project of erosion control must take account of two key ranges of requirements:

a) the general principles of erosion control,
b) the requirements of human (economic) activities in the area.

The general principles of erosion control, aimed at preventing soil erosion or reducing its intensity, include:

1. protection of the soil surface against the effect of the kinetic energy of rain drops, of the water flowing down the soil surface and of wind;
2. increase in the infiltration capacity of the soil;
3. improvement of the stability of soil aggregates;
4. increase in the retention and accumulation capacity of the soil surface;
5. less even (coarser) soil surface;
6. interception and safe conduction of erosion-triggering precipitation waters.

Erosion control must, of course, take account of the requirements relating to human activities within the region, including farming, water management, industrial operations, transport and so on.

Often several erosion-control measures have to be combined to meet these requirements; one measure in isolation (e.g., terracing) usually fails to provide adequate protection.

Erosion-control measures are classified by various criteria. There are, for example, biological and technical measures, areal and linear measures, non-investment measures and those requiring capital expenditure, agricultural and non-agricultural measures, operating and technical measures and so on.

Erosion-control measures may be included among land fund organisation efforts within site planning; with respect to this it will be useful to classify the soil fund as follows:

A. *Operating erosion control,* including:
1. anti-erosion site organisation
2. anti-erosion cultural practices
3. biological measures of erosion control;

B. *Technical erosion control* to check the effect of:
1. areal surface runoff
2. concentrated surface runoff
3. erosion activity of wind.

When planning any erosion-control project, it must be borne in mind that control measures are not alternative treatments: each of them may provide a specific solution but the required effect is usually achieved as a result of the complementary effect of several procedures.

In addition, it should be known that technical erosion-control measures (if they are considered necessary) must always be finished first although they may make no direct contribution to agricultural production. This is so because technical measures represent the necessary groundwork (mainly where the balance between the effect of erosion factors and the soil's erosion resistance was broken) on which other erosion-control measures (organisational, agrotechnical, biological) will be based. In other words, the purpose of technical measures is to stop the processes of accelerated erosion (if such processes are in progress) quickly and radically or to restore the disturbed balance between the erosion process and the soil's resistance thereto. On the other hand, the key purpose of such operations, i.e., organisational, cultural and biological anti-erosion measures, is to stabilise the balance, regardless of whether it is natural or man-made, i.e., established as a result of technical erosion-control measures.

The decision whether to take any technical erosion-control measures prior to the operating anti-erosion provisions depends on the natural conditions and on the mode of use of the land. If the field is used to produce fodder and to grow densely sown crops, no technical measures are required but in fields with root crops and other crops planted in rows wide apart, the likelihood that they will be needed is usually very high.

5.1 Anti-erosion Site Organisation

The key prerequisite for successful erosion control is a good organisation of the region. Efforts should be focused on the following four ranges of issues:
- delimitation of the land fund,
- woodland, grassland and cropland: distribution and location,
- shape, size and position of fields,
- communication network.

5.1.1 Delimitation of the land fund

Delimitation of the land fund means the division of the total area under study (e.g., watershed, farm, district, country, etc.) into "delimitation categories", based on "delimitation criteria", i.e., the region's natural conditions including its topography, soil and climate, with respect to the over-all societal interests.

By delimitation, the land fund is divided into two major categories: agricultural and non-agricultural.

a) The agricultural land fund comprises:
   - arable land,
   - permanent grassland,
   - special crop land (vineyards, hop gardens, fruit orchards and so on),
   - land temporarily not in farm use,
   - land not cultivated but necessary for farming, such as field paths, farm water reservoirs, dams to protect farmland against flooding or waterlogging, etc.

b) The non-agricultural land fund comprises:
   - forest land,
   - water areas,
   - built-up areas and yards,
   - other areas such as streets, playgrounds, gravel pits, cemeteries and so on.

For the delimitation of the land fund for erosion-control purposes, it is essential to know the criteria by which the land can be allocated to the agricultural and forest land funds.

If no over-all societal interests are involved, all land that meets the following three criteria is allotted to the agricultural land fund:

maximum inclination	25°
minimum effective soil depth	0.15 m
maximum soil skeleton content	50%

(For details see Tab. 36.)

Areas that do not meet these criteria, as well as important water-source areas, areas around medicinal springs, areas devastated by open-cast mining that cannot be reclaimed, areas designed for protective forest belts (if fruit trees cannot serve this purpose) and other areas that are to be afforested to meet the over-all societal requirements should all be allocated to the forest land fund.

On the other hand, dispersed patches of woodland on slopes of gradients of up to 12 °C with a soil profile depth of 0.6 m at the minimum and with a skeleton content not higher than 45% can be included in the agricultural land fund if they do not have a water-management, sanitation or soil-conservation function.

In delimiting the agricultural land fund, it is necessary for erosion-control purposes:

a) to distinguish areas of permanent grassland, unploughable meadows and pastures,

b) to divide arable land into "basic" and "protected" areas.

Unploughable permanent meadows include flood plains along rivers and creeks, land near lakes, ponds and other reservoirs and land in closed valleys with a high water table. As to the inclination of the terrain, the maximum gradient of slope in this category of land is 5° (in the closed valleys the maximum gradient is 8°).

Permanent pastures are, as a rule, located on hillsides with slope gradients of 17° to 25°; in valleys on slopes with gradients above 8°.

From the delimitation requirements for the forest land fund and for permanent grassland it follows that land meeting the following four criteria can be included in arable land:

- inclination of the land 17° max.
- effective soil profile depth 0.3 m min.
- skeleton content in the soil 50% max.
- altitude above sea level 800 m max.

From the viewpoint of the need of protection against erosion, arable land can be divided into two types, basic (nonprotected) and protected, depending on inclination:

1. basic arable land: slopes up to a gradient of 12°;
2. protected arable land: gradient between 12 and 17°.

More detailed delimitation criteria take account of the texture of the soil and the permeability of the subsoil, in addition to the inclination of the slope, effective depth of the profile and the skeleton content.

Table 36

A survey of delimitation categories for agricultural land in the Czech Republic

	Land fund				
	Farm land				Forest land
a	0°–20° or				20° or
b	0°–25°				25°
	Arable land		Permanent grassland		
a	0°–12° or		12°–20° or		
b	0°–17°		17°–25°		
	Basic		Protected		
a	0°– 4° or		4°–12° or		
b	0°–10°		10°–17°		
	Non-endangered	Slightly endangered	Medium endangered	Highly endangered	
a	0°–2°	2°– 4°	4°– 8°	8°–12°	
b	0°–7°	7°–10°	10°–15°	15–17°	

a – heavy textured soils $H_{S,r} > 750$ mm
b – light-textured and medium-heavy soils $H_{S,r} \leq 750$ mm

5.1.2 Woodland, grassland and cropland: distribution and location

Another range of issues important for anti-erosion site organisation includes the distribution and location of crops and stands (woodland, grassland, cropland) within the region studied which are functions of the territory's topography and hydrology and the erosion-control effectiveness of crops.

Considering the activity of water as an erosion factor, there are three distinct topographic configurations (Fig. 49): ridges, hillsides and valleys.

A ridge usually has soils of a coarser texture and a good infiltration capacity. As a result, surface runoff and water erosion are lower in such locations.

Coarse soils have a low retention capacity for water; the capillary rise of groundwater in these soils is limited; the groundwater table is usually very deep: for all these reasons the ridge locations (areas along the watershed divide) are suitable environments only for deep-rooting vegetation, especially forests. Where the climate (particularly the temperature and precipitation) is favourable, fruit trees (orchards) may be grown in the ridge areas.

Deep-rooting woody species increase the infiltration capacity of the soil, i.e., they support the change of surface water into subsurface water, thus reducing the intensity of water erosion and controlling the water regime on the hillsides and in valleys.

As to hydrological conditions, hillsides can be divided into two parts, upper and lower. In the upper part, the conditions are similar to those of a ridge: the soils have a coarse texture and a good infiltration capacity. In the lower part (usually starting in the mid-part of the hillside) the infiltration capacity is poorer and heavier rains trigger surface runoff. In the bottom part of the slope the infiltration is usually very low with surface runoff after any rain.

As said above, slope gradient and length have a significant influence on erosion intensity; hence, the location of the crop or grass or tree stand on hillsides has to be adjusted according to the slope's topography, of course with respect to the delimitation criteria for the different crops (See Tab. 38).

Valleys, or the lowlands beneath hillsides, have heavier soils whose infiltration capacity is poorer owing to the accumulation of the soil particles eroded

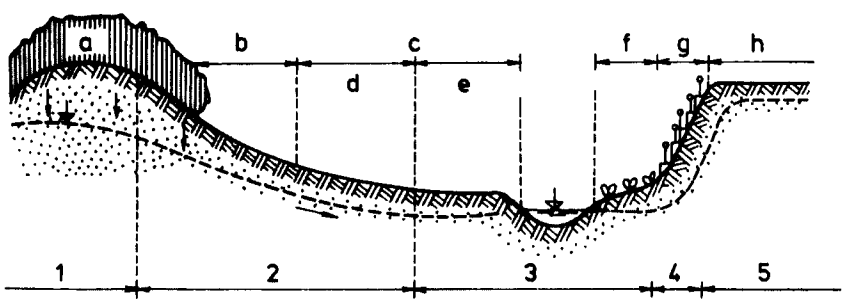


Fig.49 Division of the relief of a given area from the viewpoint of the erosion effect of water (after CABLÍK and JÚVA, 1963): a — forest, b — permanent grassland, c — crop rotation, d — arable, e — fodder crops, f — vegetables, g — orchards, vineyards, h — fields; 1 — watershed divide, 2 — slope, 3 — valley, 4 — terrace-like slope, 5 — plateau.

and transported from the slopes. They are usually well supplied with water, partly from the surface runoff down the hillsides, partly from the capillary rise of groundwater to the root zone of the soil profile. In some cases (when the groundwater table is high or the surface runoff collects in a large area or the runoff conditions are unfavourable and so on) the soil may become waterlogged.

Hence, such areas are very suitable for the cultivation of crops requiring much water such as fodder crops and vegetables; permanent grasslands will thrive in locations with an elevated groundwater table.

If in valleys or lowlands beneath hillsides the water regime of the soils is controlled, such locations can be included in the arable crop rotations.

A survey of the distribution and location of cropland, grassland and woodland with respect to the territory's topography and other characteristics is shown in Tab. 37.

Table 37

Optimum distribution and location of cropland, grassland and forest land with respect to the topography of the territory (DÝROVÁ and ŠOUKALOVÁ, 1978)

Site conditions					Suitable crops	
Terrain		Soil	Ground water table	Erosion processes	General characteristics	Crops, stands
watershed	steep	coarse-grained	deep	denudation	deep-rooting perennial	mixed forest
	plateaux	medium heavy	medium deep	mild	not very resistant to erosion	field crop rotation (potatoes)
slopes	gentle	lighter-textured (denuded)	medium deep	transport	more resistant to erosion	arable land, meadows
	steep	light-textured (strongly denuded)	deep	denudation transport	deep rooting perennial	orchards, vineyards, forests
valley	flat	heavy-textured accumulated	high (inundation)	sedimentation	shallow rooting	fodder crops, meadows, vegetables

5.1.3 The shape, size and position of fields

The delimitation of the land fund and the distribution and location of crops and grass and forest are followed by another stage, the design of the shape, size and position of fields within the area with due respect to the requirements of agricultural production (e.g., use of large-scale technology of production: mechanisation,

homogeneity of the tracts of farm land, economic aspects of crop production and so on).

The best field shape generally is a rectangle with a width-to-length ratio of 1 : 2 to 1 : 3, exceptionally up to 1 : 6, the longer side being oriented in the direction of cultivation, i.e., either along the contour lines to control water erosion or perpendicular to the direction of prevailing winds to control wind erosion.

Fields may also have a trapezoidal shape (Fig. 50), mainly in areas with uneven terrain: the interior angles of the trapezoid should not be more acute than 50°.

Other field shapes (triangles, squares, rectangles in which the ratio of sides is higher than 1 : 6 and others) are not suitable mainly because of difficult cultivation.

An example of correct positions of fields in a hillside terrain, respecting the requirements of erosion control, is shown in Fig. 51.

Field size is not only controlled by erosion factors but is also influenced by the effectiveness of the use of farm machines and by the terrain features forming permanent boundaries such as forests, roads, rivers, etc.

As far as the field has an optimum position from the viewpoint of erosion control with its longer side parallel or perpendicular to the contour lines, it should not exceed the value of the active length of the anti-deflation bands located along the longer side of the field.

As to the cultivation and use of the land, the width of the field should be a multiple of the working width of the machines used.

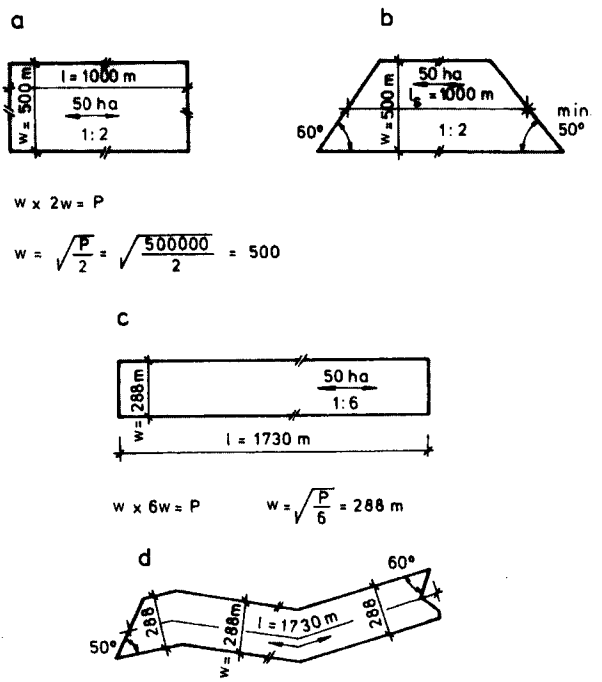


Fig.50 The best field shapes (after DÝROVÁ, 1974).

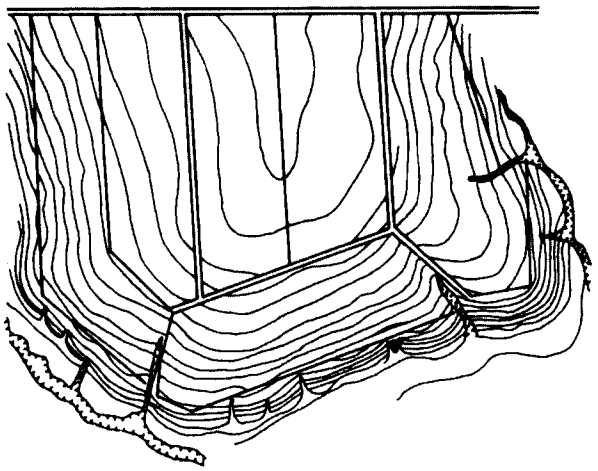


Fig.51 An example of an adequate distribution and location of fields on a sloping terrain (after CABLÍK and JŮVA, 1963).

It follows from what has been said above that the width of the field must meet the following conditions:

$$s = An \leqq L_p \quad \text{or} \quad L_u \tag{5.1}$$

where s – the width of the field (m),
A – the working width of the machines (m),
n – an integer
L_p – admissible length of slope (m),
L_u – active length of the antideflation band (m).

Field length is controlled by the economic aspects of mechanised cultural operations, including the efficiency of use of the machines (Tab. 38). A distance of about 1000 m is considered as the optimum field length (it should never be shorter than 500 m).

Table 38

Effect of field length on the effectiveness of the work of farm machines (RYBÁRSKY, 1980)

Field length (m)	Ploughing	Sowing	Harrowing, disking	Harvesting operations
200	0.70	0.76	0.73	0.85
300	0.76	0.80	0.78	0.88
400	0.82	0.85	0.83	0.90
500	0.86	0.90	0.87	0.92
700	0.90	0.92	0.89	0.93
1000	0.92	0.93	0.90	0.94
1500	0.94	0.95	0.93	0.96
2000	0.95	0.96	0.95	0.98

Field length and width are factors underlying field size and the division of fields into strips in cases of strip cultivation (see Section 5.3.2).

Since the admissible slope length depends, first of all, on the slope gradient, the latter will be among the factors controlling field size and the width of the strips in strip cultivation systems.

The guide values of field size as depending on slope gradient and on the natural conditions are shown in Tab. 39; it should be mentioned in this context that when proposing the width of a field it should be borne in mind that the conditions of equation (5,1) should also be met.

It may happen when the shape, size, location and position of a field is determined that erosion-control requirements cannot match the economic requirements for the use of the soil: in such cases a change in the current pattern of the use of the land will have to be considered (a change in technology, in the crop grown and so on).

Table 39

Effect of slope gradient on field size (RYBÁRSKY, 1980)

Gradient		Area of field in ha		
°	%	maximum	average	minimum
2	3.5	150	75	40
3	5.2	57	35	25
7	12.3	29	20	16
10	17.4	16	12	8
12	21.3	11	8	5
15	26.8	6	4	2
17	30.6	3	2	1

5.1.4 The communications network

A well-designed network of field paths and roads may serve not only for transport but also for erosion control: like natural and man-made waterways, paths and roads are effective runoff regulation factors. On the other hand, a negligently designed network may be conducive to disastrous soil erosion, e.g., with the formation of gullies.

Depending on location, field paths may be divided into ridge, contour and valley-line types. The first-mentioned, i.e., those built on the ridges along the watershed divide line, are more or less safe from erosion and relatively easy to build and maintain; this means, at the same time, that such paths have the lowest erosion-control effect: they have almost no catchment area and the amount of precipitation water that falls straight onto them is very small and easily dispersed on both sides of the paths.

On the other hand, those at the bottoms of valleys have the largest catchment area and surface water runs towards them from both sides. For this reason, ditches must line such paths on both sides to safely convey surface water not only from the surface of the path itself but from the whole catchment area.

From the erosion-control point of view, a path along the valley line with well-built ditches may replace the valley-bottom bands or the anti-erosion flow deflection channels: thus it becomes an important part of the whole complex of erosion-control measures.

On the other hand, a road incorrectly designed and built on the valley floor creates conditions (concentration of surface runoff, solifluction, mechanical damage caused to the soil by the means of transport, etc.) conducive to the formation of potholes and erosion gullies and ravines.

Of course, it often happens that requirements for transport cannot be met by building roads or paths along the ridges or valley bottoms and roads have to be built on the hillsides themselves. To control erosion, such roads should follow the contour line.

The contour-line field paths should be lined by a protective ditch on the uphill side. They should either be parallel to the contour lines or may slightly decline (5% at the maximum), remaining, more or less, within direction of the contour lines.

Paths with an incline of 0.2 to 1% are suitable both for transport and for conveying the intercepted surface water; the protective ditch usually needs no surface reinforcement (this of course also depends on the size of the catchment area).

Paths with an incline of 1 to 5% are still suitable for transport but their ditches will need reinforcement.

The distance between contour-line paths should not exceed the admissible length of slope, unless special erosion-control measures are taken between them.

Paths crossing the slope at a gradient higher than 5% are problematic from the erosion-control point of view. If a path is required to climb up the hillside, the zigzag line is usually the best solution; it may also be advantageous to combine long segments of a low gradient with short steep segments (Fig. 52).

When marking out the road network in an area with a broken relief, it is advisable to use a three-dimensional model of the territory based on stereoscopic aerial pictures. In areas where the land is endangered by erosion, it is advisable to build a system of "roundabout contour roads" (Fig. 53).

In areas for which erosion-control forest strips are designed, the best location of the field paths is on their southern, southeastern or southwestern sides: a southern

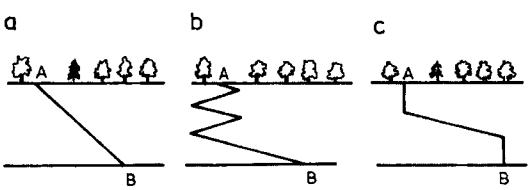


Fig.52 A general design of road on a slope (after HUDSON, 1971).

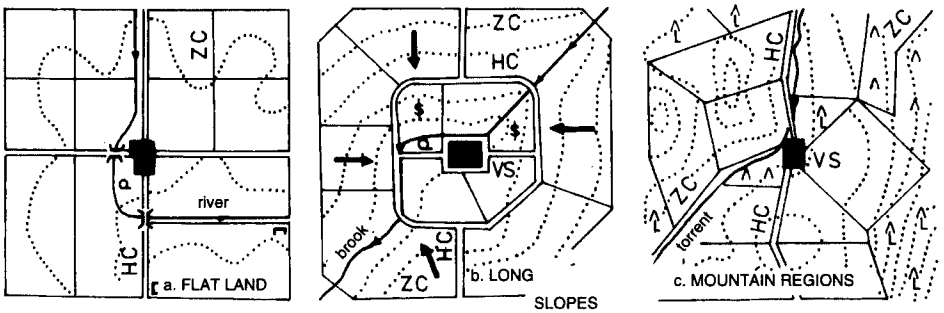


Fig.53 System of a farm-road network (after DYROVÁ, 1974): VS – farm centre, ZC – base road, HC – main road.

exposure allows the field paths, which are usually unpaved, to dry rapidly and become passable again soon after rain.

Where the forest strips stretch in the north-south direction, it is recommended to build the paths on their windward side.

5.2 Cultural operations for erosion control

In this book, cultural operations for erosion control include erosion-control measures relating to soil cultivation. These comprise, in particular, contour cultivation, erosion-control furrows and erosion-control cultural practices.

5.2.1 Contour cultivation

The contour cultivation of the soil, with all cultivation practices following the direction of the contour lines, is the key erosion-control measure used on arable land. Its application reduces water erosion intensity by up to 40%.

The reduction of water erosion intensity through contour cultivation depends on two major factors: the gradient of slope and the crop grown. Effective erosion control is obtained on slopes of a 2–12% gradient where the crops are planted in rows widely apart (especially root crops). On slopes of a gradient of 12 to 18% the contour system has a very low effect, the maximum reduction of erosion intensity being 10%. On slopes of a more than 10% incline the system has no practical importance for erosion control. This is so because at a higher gradient of slope the accumulation capacity of the furrows declines (Fig. 54).

Another factor on which the accumulation capacity of furrows depends is their size. Those for densely sown crops such as cereals, legumes, oil-bearing crops and others are very small and are visible in the field only until the first rain when they disappear and lose their erosion-control effectiveness. On the other hand, furrows for crops planted more widely apart (mainly potatoes) are usually larger and their erosion-control effect is maintained longer.

If there is a danger that strict contour ploughing may be conducive to waterlogging (this may happen in heavy-textured soils), such plots are recom-

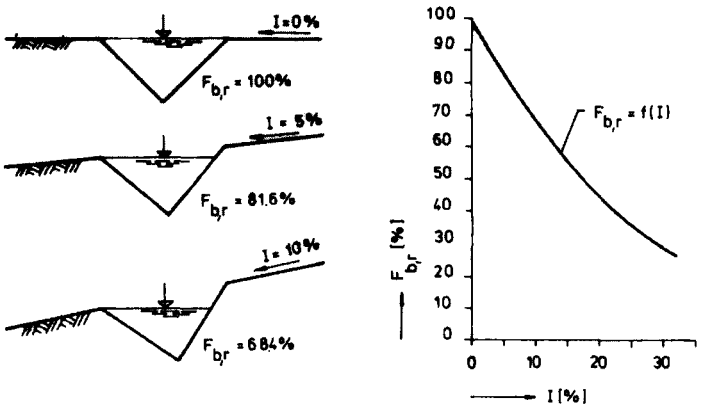


Fig.54 Effect of slope gradient on the accumulation capacity of contour furrows, $F_{b,r}$ — relative interception capacity of the furrow, I — slope gradient.

mended to be ploughed slightly aslant, crossing the contour lines: excess water may safely run along such furrows away from the field.

Data on the reduction of erosion intensity, the increase in the water reserve in the soil and the reduction of surface runoff as a result of contour ploughing are all shown in Tabs. 40 and 41.

Table 40

Effect of the method of soil cultivation on water erosion and on soil water reserves (CABLÍK and JÚVA, 1963)

Method of cultivation	Amount of soil washed away ($t \cdot ha^{-1}$)	Soil erosion ($t \cdot ha^{-1}$)	Soil moisture ($m^3 \cdot m^{-3}$)
Ploughing along slope	440.7	388.4	21.30
Unbroken stubble	74.0	46.1	23.99
Contour ploughing	93.6	13.3	29.86

As shown by the data in the tables, contour ploughing reduces soil erosion intensity almost thirty times, increases soil-water content by more than 40% and reduces surface runoff more than 13 times.

The erosion-control effectiveness of the contour cultivation of the soil can be increased by "furrowing tillage", i.e., by making furrows up to 0.5 m deep about 0.8 m apart, using a specially adapted plough. To prevent water from running along these furrows, they should be broken by levelling, using a smoothing harrow, every 50 to 200 m. A hillside treated in this way holds water, snow and eroded soil perfectly.

Table 41

Effect of terrain slope on the structure of crop rotation (CABLÍK and JÚVA, 1963)

Method of ploughing	Area of land (m^2)	Level of rain (mm)	Level of surface runoff (mm)	Runoff coefficient
Along slope	2 000	33	1.2	0.040
Along contours	3 000	31	0.1	0.003
Along contours	10 000	56	0.0	0.000

Both contour ploughing and furrowing tillage also protect the soil against wind erosion through making the soil surface coarser and increasing its moisture content; both measures also help to maintain an even snow cover, thus reducing the exposure to dry frost and preventing the destruction of the upper layer of the soil profile.

5.2.2 Furrowing

Furrowing is another option in erosion control on hillsides; it is based on making individual furrows along the contour lines at a certain distance from one another.

Furrowing, suitable also for permanent grassland, is primarily recommended for dry areas where serious erosion of the soil is caused by sporadic flood rains.

As distinct from "furrowing tillage" in which the spacing of the furrows only depends on the design of the special plough, in furrowing the optimum spacing can also be determined by calculation.

When calculating the spacing (axis-to-axis distance between the furrows) a certain number of data can be introduced in the calculation, thus improving the accuracy.

The key initial data for computing the furrow spacing include the size (dimensions) of the furrows, the gradient of slope and the characteristics of design rain.

The supporting data for improving the accuracy of the calculation include the volume coefficient of runoff, intensity of water infiltration in the furrow and the degree of precision with which the furrows are made.

For the calculation of the furrow spacing (Fig. 55) the following formulae can be used:

$$L_{d,b} = \frac{F_b}{H_{S,n}}, \tag{5.2}$$

$$L_{d,b} = \frac{F_b}{\varphi_o H_{S,n}}, \tag{5.3}$$

$$L_{d,b} = \frac{F_b + F_{i,b}}{\varphi_o H_{S,n}} a, \tag{5.4}$$

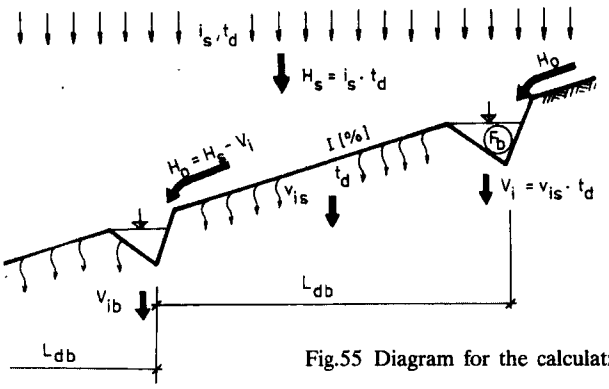


Fig.55 Diagram for the calculation of furrow spacing (after ANTAL, 1985).

where $L_{d,b}$ – furrow spacing (m),
F_b – area of the catchment (accumulation) profile of the furrow as a function of furrow shape and size and slope gradient (m^2),
$H_{S,n}$ – ($= i_{S,n} t_d$) level of simulated rain (m),
t_d – length (duration) of design rain (s),
φ_o – runoff volume coefficient,
$F_{i,b}$ – ($= v_{i,s} t_d o_b$) extension of the area of the catchment profile of the furrow through inclusion of the infiltration of water via the furrow bottom and walls (m^2),
$v_{i,s}$ – mean value of the intensity of water infiltration through the furrow bottom and walls during the design rain ($m \cdot s^{-1}$),
o_b – mean value of the wet perimeter of the furrow during the course of design rain (m),
a – ($\leqq 1$) coefficient expressing the perfection of the furrows.

When calculating the spacing of the furrows it should be borne in mind that a safe spacing design (from the viewpoint of erosion control) is obtained when the runoff volume coefficient and the water infiltration along the wet perimeter of the furrows are left out, i.e., on the assumption that $\varphi_o = 1$ and $F_{i,b} = 0$.

On the assumption of a perfect formation of the furrows (for $a = 1$), the design for furrow spacing is less safe from the viewpoint of erosion control. These conclusions are generally applicable in designing erosion-control measures.

Another option is to dig holes in the soil surface. This measure has the same erosion-control effect as furrowing, including the interception of the water running down the surface and transferring it into ground water and the interception of eroded soil particles.

The holes (depressions, mini-reservoirs) made on the soil surface may be of different sizes and are not connected with one another; they are dug either by means of special hole diggers or by intermittent or cross furrowing. The former entails making cross furrows (spacing 3–4 m) in the field, followed by building small dams 4–5 cm on the furrows. The latter is more effective: by this method furrows are ploughed 1–2 m apart in two directions, one crossing the other.

Furrowing and hole digging are erosion-control measures applicable only on surfaces not exposed to the inflow of surface waters from outside.

5.2.3 Cultural practices for erosion control

Research and practical applications in cultural operations designed with respect to the need of erosion control have enjoyed much attention in recent years. Despite all research, however, there is one simple rule of thumb applicable in all situations: this rule says that any measure taken to increase the production of biomass is at the same time a measure to control erosion. In other words, there is no antagonism between increasing soil fertility and erosion control.

From this viewpoint, any fertilisation, soil water regulation, chemical treatment, introduction of high-yielding varieties and strict observation of optimum dates of cultural operations, as well as the quality of the cultural operations themselves, can in fact all be considered as belonging among erosion-control measures.

Cultural anti-erosion operations must improve a number of parameters in addition to increasing soil fertility. These parameters include:

- infiltration capacity of the soil,
- erosion resistance of the soil,
- soil surface coarseness,
- effective management of soil water,
- soil surface protection from the effects of water and wind.

The key measures to meet these requirements for the soil include:

- organic manuring of the soil,
- conservation tillage,
- use of plant residues,
- regulation of the water regime of the soil.

Application of organic manure (e.g., farmyard manure, compost, peat, green manure, straw) improves not only soil fertility but also the infiltration capacity of the soil, its erosion resistance and the utilisation of the soil water.

The favourable effect of the presence of organic matter in the soil includes its

- improving the biological properties of the soil,
- improving the sorption capacity of soil particles,
- enriching the soil with carbon and nitrogen,
- improving the hydrophysical properties of the soil,
- improving the thermal regime of the soil,
- enabling a better development of the root system (IVANIČ et al., 1979).

The combination of organic manuring with liming provides conditions for the formation of a fine, lumpy, water-resistant and porous soil texture (SOTÁKOVÁ, 1979); hence, conditions are created under which the soil can acquire optimum characteristics for a higher erosion resistance, for a higher infiltration capacity and for a better utilisation of soil water.

Conservation tillage is to provide an improved infiltration capacity and erosion resistance of the soil, a coarser soil surface and more effective utilisation of water; in other words, it can in fact meet all the requirements for the erosion-control function of cultural operations. The problem is that conservation tillage requires special machinery which is not commonly available.

In addition to the common cultural operations such as deep tillage, stubble breaking, smoothing, harrowing, rolling and others, conservation tillage includes the following (WISCHMEIER, 1970):

a) *Minimum soil cultivation* in which tillage and sowing are combined in a single operation. Minimum soil cultivation leaves a coarse soil surface with fine lumps between the rows of cereal crops, the soil texture being highly porous. This system of soil cultivation provides a much higher accumulation capacity of the soil surface, compared with the traditional system of seedbed preparation for cereals (e.g. tillage, disking, harrowing, sowing). Minimum soil cultivation reduces the average surface runoff of precipitation water by 35% and soil erosion by 40%.

b) *Subsurface cultivation system*, or no-tillage system. This system leaves the soil surface practically intact: the only treatment is the undercutting, loosening and drying of the upper soil layer to kill emerging weeds without cutting the plant residues (stubble) on the surface. Various types of undercutters (Fig. 56) have been developed for this treatment: arrow-shaped blades are the best for water-erosion control, chisel-like blades are the best for wind-erosion control.

When the soil is free of weeds (e.g. after maize or after herbicidal treatment) even the undercutting of the soil surface may be left out and the seeds may be drilled directly in uncultivated soil, using special drills.

As shown by field experiments, there is little difference in yields obtained after traditional preseeding soil preparation and after the application of the no-tillage system, however, the latter system provides a better quality of grain and reduces soil erosion by up to 70%.

c) *Strip cultivation.* With this system the preseeding preparation is based on loosening narrow strips (width about 0.2 m, depth of about 0.1 m), preferably following the contour lines and leaving the land between the strips uncultivated. Rototillers are used in this system of soil cultivation.

The advantage of strip cultivation is that it removes the weaknesses of the no-tillage system or the subsurface cultivation: the plant residues do not hinder drilling and do not reduce soil temperature around the planted seeds. As to erosion control, strip cultivation is as effective as subsurface cutting or the no-tillage system.

In heavier-textured soils (PASÁK, 1980), good results have been recorded with using disk-type drills to plant cereals in autumn into the soil cultivated by stubble ploughing, sown with a catch crop and treated with a siccant before sowing the main crop. Spring-sown crops may also be planted with advantage in a field in which a catch crop (e.g., winter rape) grows.

Further systems of conservation tillage are being developed and the existing systems are being refined (e.g., the width of the loosened strips is reduced to 0.5−0.08 m); generally speaking, any system of conservation tillage is based on:

- minimum turning of the topsoil,
- minimum area of soil surface broken by the cultural operations,

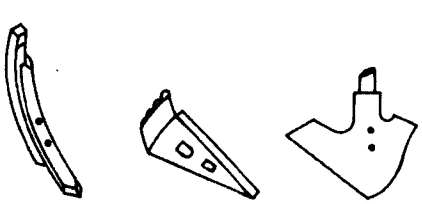


Fig.56 Main types of subsoilers for the no-tillage system of soil cultivation.

– making use of the erosion-control effect by plant residues usually by leaving them on the surface, sometimes by incorporating them in the topsoil.

Regulation of the water regime of the soil reduces the intensity of soil erosion both directly and indirectly. The indirect reduction of the intensity of both water and wind erosion is through encouraging the development of the vegetation cover of the soil. Where the soil-water regime can be regulated, it is possible not only to increase the density of the crops grown, thus enhancing the protection of the soil against the erosive activity of water and wind, but also to extend the period for which the soil is protected by the plant cover (e.g., through growing catch crops or using the two-harvests system in irrigated areas).

The direct reduction of the intensity of wind erosion is through maintaining moisture in the surface layer of the soil: while the soil surface is wet it is fairly resistant to the erosive effect of wind.

The soil moisture reserve can be increased by using the water from melting snow and, of course, by the irrigation of the soil, in addition to the agrotechnical methods of soil-water regulation which may sometimes hinder erosion control: e.g., rolling of the soil surface and leaving the surface free of vegetation may be favourable for the soil-water regime but adverse to erosion control (ZACHAR, 1984).

For a better utilisation of snow water it is necessary, first of all, to provide a regular distribution of the snow cover over the given territory (e.g., by establishing a network of forest shelter belts, by erecting snow guards, by leaving guard strips of maize or sunflower stalks in the fields, etc.) and to extend the retention area of the soil surface (e.g., through hole digging, furrowing, check damming etc.).

In soil irrigation (for details see BENETIN et al., 1979), it should be borne in mind that inappropriate management of the irrigation system may cause soil erosion. This may happen, for example, if the supply of water to the soil continues for longer than the time to the onset of pool formation (t_p) (see Section 3.2.1.3) which is a function of the infiltration capacity of the soil and intensity of spray irrigation.

For a rapid determination of spray-irrigation intensity and duration as a function of the soil's infiltration capacity and of the maximum quantity of irrigation water that does not trigger surface runoff (and thereby irrigation erosion), it is recommended to use a graphico-analytical method (ANTAL, 1988).

The method is based on graphically determining the derivation of the cumulative infiltration curve of irrigated soil at a point representing the irrigation norm (M_d), i.e., the quantity of irrigation water to be supplied to the soil (Fig. 57).

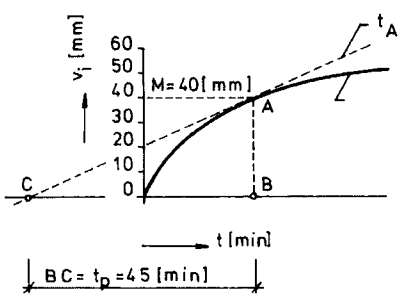


Fig.57 Graphico-analytical method for the determination of the admissible rate of spray irrigation (after ANTAL, 1988).

5.3 Biological measures for erosion control

As suggested by the heading, biological anti-erosion measures are based on the erosion-control effects of the plant cover of the soil. The vegetation cover contributes to meeting all the six principles of erosion control: it protects the soil against the kinetic energy of water and wind, increases the soil's infiltration capacity, improves the stability of soil aggregates, extends the retention and accumulation capacity of the soil surface, provides a coarser soil surface and is able both to catch and safely convey part of precipitation water.

The most important biological measures include (HOLÝ, 1978): soil conservation rotation of crops, strip cropping, anti-erosion grassing, forest shelter belts and conservation afforestation.

5.3.1 Soil-conservation rotation of crops

Soil-conservation rotation means that crops are rotated in a given field so as to provide that:

$$C_{OP} \leqq C_{\text{adm}}, \tag{5.5}$$

where C_{OP} – $(= \sum C_n/n)$ factor of vegetation cover and soil cultivation for the whole crop rotation,

C_{adm} – $(= S_{p,\text{adm}}/R, K, L, S, P)$ admissible level of the vegetation cover and soil cultivation factor for the given plot, i.e., a level of factor C at which the admissible loss of soil will not be exceeded,

C_n – vegetation cover and soil cultivation factor of the n^{th} crop in the rotation,

n – number of crops in rotation,

$S_{p,\text{adm}}$ – admissible loss of soil from the plot investigated; the admissible loss of soil is primarily a function of soil profile depth ($t \cdot ha^{-1} \cdot year^{-1}$),

R, K, L, S, P – values of erosion factors of the plot investigated.

Different groups of crops have different erosion-control potentials (Section 3.2.4.2); hence, the higher the exposure of the field to the erosive activity of water and wind, the larger should be the proportion of crops of a high erosion-control potential in the rotation (especially perennial fodder crops).

Some guide data on the required proportions of perennial fodder crops and root crops, depending on the inclination of the terrain, are shown in Tab. 42.

5.3.2 Strip cropping

With strip cropping, the soil is protected against the destructive effects of erosion factors by the alternation of strips of crops with a poor potential for erosion control (e.g., root crops and cereals) with strips of crops with a good potentional (e.g., fodder crops) (Fig. 58).

Table 42

Effect of terrain slope on the structure of crop rotation (HOLÝ, 1978)

Gradient of slope	Relative proportions of the rotation	
	perennial fodder crops %	root crops and maize %
0 – 5	9 – 10	25 – 30
6 – 10	11 – 12	13 – 17
11 – 15	15 – 18	6 – 10
16 – 17	20 – 30	
18 – 20	40 – 60	

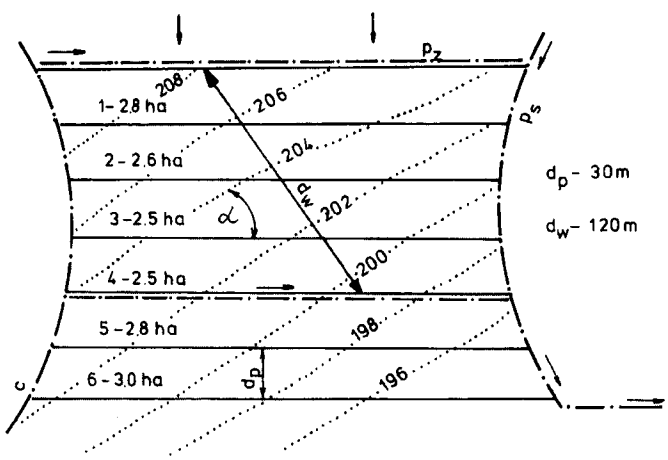


Fig.58 Diagram of strip cropping (Typification Guidelines, 1984): 2 – 6 strips, odd-numbered strips perennial fodder crops (cereals), even-numbered strips root crops, p_z – interception ditch, p_s – outlet ditch, c – road (see Fig.63).

The strip of a crop with poor erosion-control potential is called a sheltered or protected strip and that of a crop with good potential is called a shelter or buffer strip. The latter is part of the crop rotation in the given field.

Depending on the type of erosion to be controlled, there are either

– *contour shelter strips* which prevent water erosion of the soil; as far as possible, such strips should follow the direction of the contour lines, the maximum admissible deflection of the infiltration belt from the direction of the contour line being 30°,

– or *antideflation shelter strips* which prevent or reduce wind erosion; such strips should cross the direction of prevailing winds.

The erosion-control effect of the contour shelter strips rests in breaking the erosion-active surface runoff of precipitation water, in converting surface water

into subsurface water and – thereby – in protecting the nearest "downstream" crop strip.

To be able to control erosion, the shelter strip should be designed so that (Fig. 59):

$$O_{S,L} + O_{S,D} = O_{i,L} + O_{i,D} \quad \text{or} \tag{5.6a}$$

$$O_{p,L} + O_{S,D} = O_{i,D} \tag{5.6b}$$

$$L \leqq L_p, \tag{5.7}$$

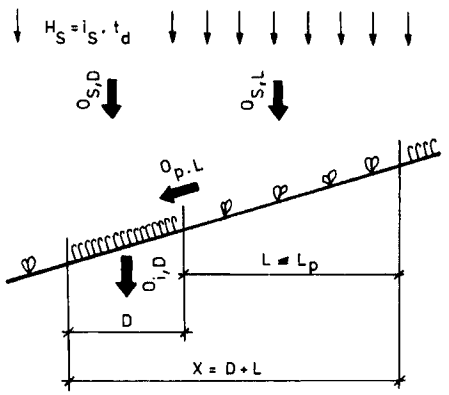


Fig.59 Diagram of the action of an anti-erosion infiltration strip: D – width of the infiltration strip, L – infiltration strip spacing.

where $O_{S,L}$ – ($= i_S t_d L \cdot 1$) volume of precipitation water that falls from design rain onto a sheltered strip of unit length (m^3),

$O_{S,D}$ – ($= i_S t_d D \cdot 1$) the volume of precipitation water that falls from design rain onto a shelter strip of unit length (m^3),

$O_{i,L}$ – ($= v_{i,L} t_d L \cdot 1$) volume of water that infiltrates into the soil during the course of design rain in a sheltered strip of unit length (m^3),

$O_{i,D}$ – ($= v_{i,D} t_d D \cdot 1$) volume of water that infiltrates into the soil during the course of design rain in a shelter belt of unit length (m^3),

$O_{p,L}$ – ($= O_{S,L} - O_{i,L}$) volume of surface runoff from the sheltered strip during the course of design rain (m^3),

L – width of the sheltered strip, i.e., spacing of shelter strips (m),

D – width of the shelter strip (m),

L_p – admissible length of slope (m),

i_S – intensity of design rain ($m \cdot s^{-1}$),

t_d – time of duration of design rain (s),

v_i — average intensity of water infiltration into the soil during the course of design rain on the sheltered strip ($v_{i,L}$) and on the shelter strip ($v_{i,D}$) ($m \cdot s^{-1}$),

Equations (5.6a) or (5.6b) can be rearranged into an equation to calculate the width of the shelter strip:

$$D = L \frac{i_S - v_{i,L}}{v_{i,D} - i_S}, \tag{5.8}$$

or the volume coefficient of runoff from the sheltered strip can be introduced; then the equation will be

$$D = L \frac{\varphi_{o,L} i_S}{V_{i,D} - i_S}, \tag{5.9}$$

where $\varphi_{o,L}$ — $\left(= \frac{O_{p,L}}{O_{S,L}}\right)$ volume coefficient of runoff from the sheltered strip.

The use of equations (5.8) or (5.9) requires knowledge of the infiltration capacity of the soil on the sheltered strip ($v_{i,L}$) and shelter strip ($v_{i,D}$) under different crops and also of the characteristics of design rain (i_S, t_d) and the admissible length of slope ($L \leqq l_p$).

The only data so far available for designing shelter belts are those on the effect of fodder crops on the infiltration capacity of the soil (Tab. 43); in view of this, a formula was drawn up for computing the width of the shelter strip, using the CN method (ANTAL, 1985):

$$D = L \frac{H_{O,L}}{0.2H_{RD} - H_S}, \tag{5.10}$$

where $H_{O,L}$ — level of surface runoff from the sheltered strip calculated by equations (3.39) and (3.42), the value of CN_L being defined as a function of actual and design characteristics of the sheltered strip (data in Tab. 17) (mm),

$H_{R,D}$ — retention potential of the shelter strip calculated by equation (3.42), the value of CN_D being defined as a function of actual and design characteristics of the shelter strip (data in Tab. 17) (mm),

H_S — level of the design rain contemplated.

If equation (5.10) is to be true, it should hold that:

$$0.2H_{R,D} > H_S. \tag{5.11}$$

Equation (3.42) can be used to represent $H_{R,D}$ in formula (5.11) which can then be rearranged:

$$CN_D < \frac{5080}{H_S + 50.8}. \tag{5.12}$$

Table 43

Effect of the growing of fodder crops on the infiltration capacity of soils (RIEDL and ZACHAR, 1973)

Soil	Sandy soil with permeable bedrock	Sandy soil with impervious bedrock	Loamy sand with permeable bedrock	Loamy sand with impervious bedrock	Sandy loam with permeable bedrock	Sandy loam with impervious bedrock	Loamy soil with permeable bedrock	Loamy soil with impervious bedrock
Intensity of infiltration $mm \cdot min^{-1}$	2.0	1.7	1.7	1.4	1.4	1.1	1.1	0.8

Formula (5.12) can be used to select the best crop for the shelter strip.

Using the nomogram (Fig. 60) it is possible to identify the crops suitable for the shelter strip as a function of the level of design rain (H_S) and the content of category I particles in the soil quickly and fairly accurately; the nomogram also allows of determing other data needed to solve equation (5.10), i.e., the CN_L value as a function of the crop grown on the sheltered strip and of soil class (content of category I soil particles).

It follows from an analysis of the equations for shelter-strip width that strip cropping (Fig. 61) as an erosion-control measure will only be successful on lighter-textured soils. For practical reasons, the width of the shelter strip usually is $D = L_p$ at which the shelter strip may not have a 100% performance: this weakness is offset by a supporting measure, e.g., by building an outlet or infiltration ditch along the lower boundary of the contour strip to prevent erosion of the downstream crop strip.

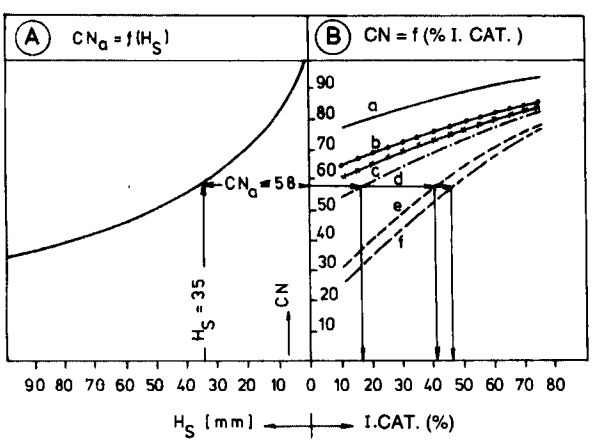


Fig.60 Nomogram for the determination of the best crops for the infiltration strip according to the level of design rain H_s and the content of category I soil particles (after ANTAL, 1986), a — fallow, b — root crops, c — cereals, d — fodder crops, e — permanent grassland, f — forest.

Fig.61 Strip cropping (Photo M. Janeček) (see Fig.60).

In areas exposed to severe erosion, shelter strips are often established as permanent infiltration belts which cannot be included in the crop rotation. Depending on the local conditions, the protective vegetation cover may consist of grass, fruit trees or shrubs and deciduous or coniferous trees. The width of these infiltration belts is determined as described above.

The erosion-control performance of antideflation protective belts is based on a reduction of wind speed near the soil surface, i.e., reduction of the wind's erosivity. With this method, strips of short-growing plants or plants with a poor erosion-control potential (i.e., sheltered strips) alternate with strips of tall plants, e.g., maize, sunflower and others (i.e., shelter [anti-deflation] strips).

The favourable anti-deflation performance of the shelter strips will be greater if the aerial parts of the shelter plants are left in the field for as long as possible (even throughout winter, if possible) and if the anti-deflation strips are part of a complex of wind erosion control measures. Anti-deflation strips cannot protect the soil immediately after they are established.

The design of the distribution of anti-deflation strips is based on the data concerning the prevailing winds and the distance controlled by the protective effect of the strips.

The distance controlled by the anti-deflation strips is the distance to which the strips can reduce the windspeed. On the lee side of the strips, this distance is 15 to 20 times the strips' height and on the windward side about 5 times their height.

5.3.3 Anti-erosion grassing

Anti-erosion grassing is primarily designed for areas (see Section 5.1.1) that are exposed to a severe threat of erosion, that cannot be farmed on a large scale, that have poor soil and climatic conditions for arable farming and those that cannot be effectively afforested.

Grassing is also recommended in erosion-exposed ravines and valley floors with large catchment areas, including also land used for arable farming (valley floor grass strips within the paths of concentrated runoff).

The erosion-controlling and stabilising effect of permanent grassland is also a good reason for grassing drift sands, unproductive land, landfills and waste dumps, open grounds within residential areas (especially in housing estates), etc. Such permanent grasslands not only control erosion but also have aesthetic and sanitary functions.

The best ways of renewal of permanent grassland designed for erosion control are rejuvenation, underseeding with cultivated grasses, additional manuring and irrigation. Ploughing should never be employed to renew such grassland.

If erosion-control permanent grassland is used for pasture, the animals should not be allowed to break the compact sward, to overgraze the grass or to tread paths.

Strip grazing is the best way to keep grazing under control and to prevent the animals from deteriorating the grass.

The species composition of the grasses and clovers should be adjusted to the use of the grassland for grazing: hardy species that can withstand grazing and treading should prevail, including primarily cocksfoot, awnless brome, canary grass, Dutch clover, lucerne, timothy and all species of meadow grass.

5.3.4 Forest shelter belts

Forest shelter belts are tree and shrub belts having a sufficient width and established at sufficient spacing to be able to control erosion.

Depending on the type of their anti-erosion function, they can be divided into windbreaks, filter belts and shading belts. The purpose and methods of establishment of forest belts are discussed in detail in Section 4.6.

5.3.5 Conservation afforestation

On steep ridges and hillsides that are heavily damaged by erosion and on unproductive land exposed to erosion, the best biological way of erosion control is to form a continuous forest stand. Conservation afforestation is also preferred for environmental reasons. Forests at such locations should be adequately managed and may be included in the special purpose category.

Depending on location, this type of erosion control can be divided into the afforestation of ridges and hillsides and that of unproductive areas.

Conservation afforestation is discussed in detail in Section 9.4.2.

5.4 Technical measures of erosion control

The anti-erosion organisation of the land fund and the cultural and vegetational erosion-control measures are combined with technical interventions to control wind and water erosion: the objectives of such technical measures are to prevent the loss of too much soil, to help in avoiding too early concentrations of surface water, improve the infiltration thereof into the soil and to reduce windspeed, all by technical means, especially through adjustments of the topography, slope gradient and slope length, through the protection of land, residential areas and special facilities against external water and through the protection of water reservoirs, channels, ditches and other structures against rapid silting. Hence, technical erosion-control measures should be included in projects for road networks, waterway regulation, irrigation systems, various structures for water management, etc.; at the same time, such measures should adequately combine with the cultural anti-erosion operations.

The technical measures for water erosion control include terrain levelling, terrace channels, ditches, check dams, terraces and erosion-control reservoirs. The success of erosion control depends on the durability and service life of the technical features involved which, in turn, depend on the adequacy of solutions to hydrotechnical issues in the erosion-control project (dimensions of structures, channel stabilisation, projected safety level, etc.), on adherence to project parameters during construction and on regular maintenance and optimum management after completion of the project.

5.4.1 Terrain regulation

Land in uneven terrain, especially in areas with cross undulation, is exposed to the danger of the concentration of water from flood rains and snowmelt along the valley line. On very deep soils, especially loess, where the gully or scour erosion process is starting, the terrain can be regulated to form an inclined plane or a convex slope and to reduce the slope gradient thereby. Terrain regulation reduces the unevenness of the terrain (microrelief); sometimes it may also reduce the length of the slope, improve access to a particular field or unify scattered patches and, in particular, improve the runoff of surface waters.

Terrain levelling is recommended to level out fields with height differences of up to 10 cm and untreated slope gradients of 15 to 21% in large-scale projects; if undulation of local importance is to be levelled, the gradients of slope may be greater. The resultant gradient is designed with respect to the type of culture (arable, orchard, vineyard) and should not exceed 18%, exceptionally 21%. The measures should never obstruct the paths of concentrated flow, so terrain levelling is, as a rule, combined with other erosion-control measures.

Terrain regulation measures may also include the design of isolated boundary banks, reinforced by retaining walls where necessary. Some boundaries and banks are sometimes removed during the levelling operations but this is only done when it has been found that their erosion-control performance is poor and their environmental importance low. Clearance of abandoned hollowed-out farm tracks and

similar depressions, especially those overgrown with woody species, also requires environmental assessment.

With any terrain levelling, the topsoil must first be removed to be put back when the earthwork is finished; this should be followed by biological reclamation.

5.4.2 Terrace channels

Terrace channels are shallow and wide and their slopes have a low gradient. They either intercept and infiltrate the water running down the surface (terrace interception channels) or safely convey the water (terrace outflow channels). Terrace channels do not hinder soil cultivation: they can be passed by farm machines and sown to crops. The interception space of the terrace channels can be extended by a low dam on their downgrade side. The water from terrace channels runs to ditches or small streams: discharging the water from the terrace channels into pipes is not recommended.

The interception terrace channels (Fig. 62) can be built in plots at gradients of 5−15%, exceptionally 18%; at higher gradients (12% and more) such channels are only designed for areas where the soils are deep and they can never be built in areas exposed to the danger of landslides. To have a good performance, the terrace channels should be built at an appropriate spacing and must have an adequate retention space. Built along the contour line, they have no inclination and if they do incline the gradient should not exceed 3%. The maximum admissible distance between two neighbouring terrace channels is calculated by the following equation (see Section 4.3.2):

$$L_p = \frac{G_{\text{adm}}}{RKSCP} \quad \text{(m)}. \tag{5.17}$$

The volume of the retention space is theoretically based (JŮVA and CABLÍK, 1954) on the assumption that the terrace channel should be able to hold all the intercepted water with no spill over the channel's bank. Hence,

$$hb_s = \varphi i_{S,n} t_d L_p \quad \text{(m)}, \tag{5.18}$$

where h − depth of the terrace channel (m),
b_s − mean width of the terrace channel (m),
φ − runoff coefficient (see Section 3.1.2),
$i_{S,n}$ − intensity of design flood rain ($\text{m}\cdot\text{s}^{-1}$) (Section 3.1.2),
t_d − time (duration) of design rain (s) (see Section 3.1.2),
L_p − admissible length of slope (m).

Terrace channel slopes are recommended to have gradients of 1 : 10 to 1 : 15; the recommended depth h is 20 to 100 cm and the length 600 m.

Interception terrace channels are usually built without reinforcement (on arable land); shallow ones are cultivated, whereas the deeper ones with steeper walls are grassed. They are either parallel or non-parallel to the contour lines. Parallel channels are easier to cultivate mechanically. Interception terrace channels built with no inclination on heavier-textured soils may sometimes fill with silt which is

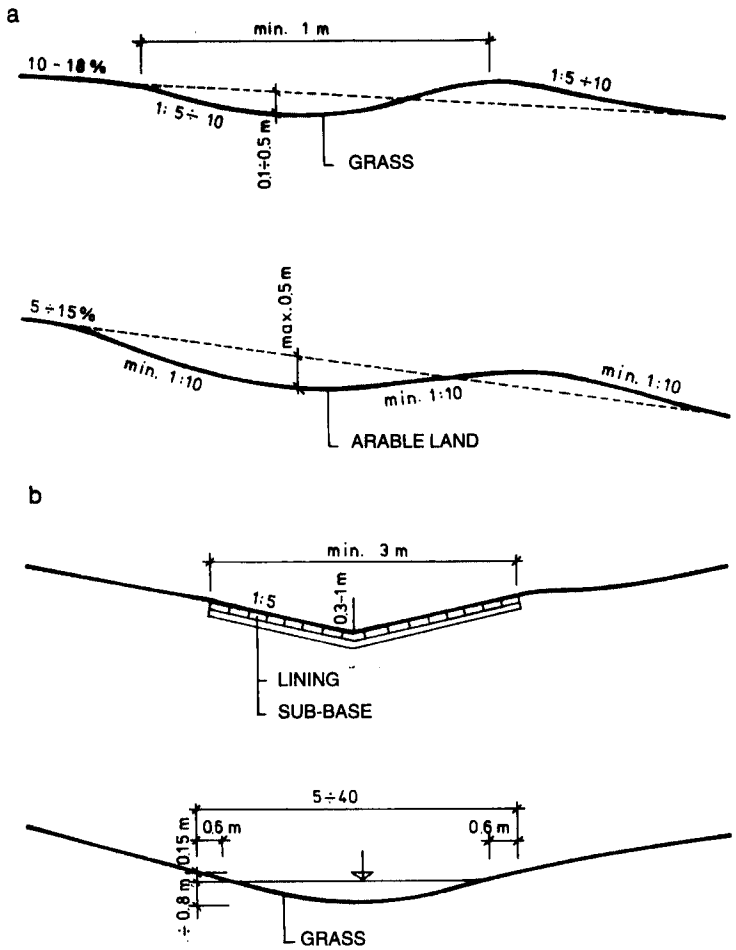


Fig.62 Anti-erosion terrace channels: a — interception terrace channels, b — terrace outlet channels.

often muddy, hindering the use of heavier machinery and damaging the plants. Hence, it is helpful to convey the water along a slight incline or to install a drain along the axis of the terrace channel. Crossing the channels with farm machines should be avoided because the wheel tracks would reduce the erosion-control effectiveness thereof. Terrace channels overgrown with grass are suitable in permanent grasslands, in filter strips, in strip cropping and other systems (see Section 5.3). The cross profile of inclined grassed terrace channels is designed as in terrace outlet channels.

Interception terrace channels are easy to build, using multishare ploughs to make an open furrow on the bottom and a ridge on the dam (see Fig. 62). The maintenance of cultivated terrace channels is based on the same principle. Fields with terrace channels are best ploughed with reversible ploughs; other cultural operations including seeding, planting, harvesting, etc. should also be performed with respect to the location of the terrace channel (for details see SEDLÁK, 1976).

Terrace outlet channels convey the short-duration flows from flood rains and spring snowmelt. On arable land, for the most part, they are built on valley floors; in orchards and vineyards they form a part of paths. The preferred cross profile is parabolic but may also be triangular (along the paths), the walls inclining 1 to 2%, exceptionally more.

The dimensions of the cross profile are calculated on the assumption of a uniform maximum flow of a design (usually 10-year) flood, using the CHÉZY & MANNING equations. The procedure of the calculation is shown, for example, by SANETRNÍK (1984) and the tables of the key parameters of grassy terrace channels of triangular cross profile are published by PASÁK et al. (1984) (Tab. 44). The simplified calculation after ZUNA (1987) can also be used with advantage (Tab. 45). The profile shape (parabola, triangle) is chosen for the design flow rate, determined as described in Section 4.1.2, and the gradient of the channel (I) is then determined according to the actual conditions. The limit depth of water, y_m, and the limit flow quantity, Q_m, are computed by the following formulae:

$$y_m = D : I \qquad \text{(m)}, \tag{5.13}$$

where I – the gradient (longitudinal inclination) of the terrace channel,
D – constant shown in Tab. 45.

and

$$Q_m = A y_m^{2.67} I^{0.5} \qquad (\text{m}^3 \cdot \text{s}^{-1}), \tag{5.14}$$

where A – constant shown in Tab. 45,
y_m – limit water depth (m),
I – the gradient (longitudinal inclination) of the terrace channel.

If the design flow rate, Q_m, is lower than the maximum (culmination) flow, the design will be safe; otherwise the gradient I has to be reduced

Table 44

Basic parameters of a grassed channel with a triangular profile and a slope gradient of 1 : 5 (DÝROVÁ, 1984)

Depth	I = 1%		I = 5%		I = 20%		I = 100%	
	v	Q	v	Q	v	Q	v	Q
m	$\text{m} \cdot \text{s}^{-1}$	$\text{m}^3 \cdot \text{s}^{-1}$	$\text{m} \cdot \text{s}^{-1}$	$\text{m}^3 \cdot \text{s}^{-1}$	$\text{m} \cdot \text{s}^{-1}$	$\text{m}^3 \cdot \text{s}^{-1}$	$\text{m} \cdot \text{s}^{-1}$	$\text{m}^3 \cdot \text{s}^{-1}$
0.1	0.141	0.0071	0.316	0.0158	0.631	0.0316	1.340	0.070
0.2	0.224	0.0448	0.501	0.1002	1.000	0.2004	2.130	0.430
0.3	0.294	0.1321	0.657	0.2955	1.313	0.5910	2.800	1.260
0.4	0.356	0.2846	0.796	0.6364	1.591	1.2727	3.390	2.710
0.5	0.413	0.5159	0.923	0.1537	1.846	2.3074	3.930	4.910

Note: At $v \geq 1.3\ \text{m} \cdot \text{s}^{-1}$ the bottom must be stabilised by sills and by a stronger type of lining than sod.

Table 45

Values of constants *A, B, C, D* (ZUNA, 1987)

Constant	Triangular profile			Parabolic profile		
	gradients of slopes			*b* : *y* ratio		
	1 : 3	1 : 5	1 : 7	12	20	28
A	45	78	109	152	253	356
B	15	16	16	19	19	19
C	7130			11 660		
D	0.0112			0.0068		

and the calculation repeated. Further, the mean cross-sectional velocity of the water and the maximum tangential stress should be assessed:

$$\tau_m v = B y_m^{0.67} I^{0.5} \qquad (\mathrm{m \cdot s^{-1}}), \tag{5.15}$$

where B – constant to be found in Tab. 45,
y_m – limit water depth (m),
I – gradient (longitudinal inclination) of the terrace channel;

and

$$\tau_m = C y_m I \qquad (\mathrm{Pa}), \tag{5.16}$$

where C – constant in Tab. 45,
y_m – limit depth of water (m),
I – gradient (longitudinal inclination) of the terrace channel.

The limit values of tangential stress are shown in Tab. 46. The data in the table will indicate the need of an adequate reinforcement of the terrace outlet channels at higher inclination values and higher water velocities, especially when the water has to be conveyed frequently and at high flow rates. It is useful to combine the conveyance function of the channels with field paths (Fig. 63). Some harder reinforcement of the terrace channels may be necessary in places with higher flow rates (on terrace chutes), on the passes crossing the terrace channels, on headlands, etc. Terrace outlet channels and grassy strips in hollows are recommended to be built with a drain but the drain should not follow the valley line because this may lead to the formation of an erosion gully. The drain will eliminate the danger of damage caused to the bottom of the terrace channel by passing machines or cattle.

Bulldozers or graders are used to build the terrace outlet channels, though multishare ploughs or disk harrows will sometimes suffice to do the work. Grassed terrace channels require regular fertilization, mowing and harvesting, and also regular inspection followed by filling the developing gullies with

Table 46

Values of limit tangential stress τ for different types of lining

Type of lining	Pa
Sod-grass, not mown	70
Sod-grass, mown	80
Lawn	90
gravel d_e = 0.10 m	100
gravel d_e = 0.13 m	120
gravel d_e = 0.15 m	130
gravel d_e = 0.18 m	150
pole fence	150
slabs 30 × 60 × 10 cm	150
"Klas" Blocks	160
Vegetation blocks IBM, IZT	200

Fig.63 Field path serving as a terrace outlet channel (Photo R. Hejl).

earth, compacting the filling and putting pieces of sod on top. Tillage in the proximity of the terrace channel should cross the valley line to prevent the formation of gullies.

5.4.3 Ditches

Erosion-control ditches are used for the protection of downstream fields or structures against surface runoff or concentrated flow which may often

carry eroded soil. As to their function, there may be peripheral interception ditches, collection interception ditches and outlet ditches.

The peripheral interception ditches catch water from agricultural, forest or other areas to protect downstream farm land, buildings, residential areas, water management facilities (mainly in the hygienic protection zones), etc. The collection interception ditches are built inside the protected areas; they have a slight inclination and their spacing (as with the interception terrace channels) on longer slopes depends on the admissible slope length L_p. Water and eroded earth from the interception ditches and terrace channels are conveyed along the outlet ditches. These usually have a high gradient, so they must be adequately reinforced. Channels lining roads may serve the same purpose.

Ditches have two major weaknesses: the land they take up cannot be cultivated and they cannot be crossed by farm machines. Pipe culverts have to be built in the ditches. Sedimentation pits are built to catch the soil carried by the water; for safe conveyance of the water, the ditches should be fitted with inlet facilities, chutes, drops, etc.

In designing erosion-control ditches, the key requirement is that the ditch should be able to convey the design flow (see Section 3.1.2); the collection ditches should also have a certain flow speed (0.5 $m \cdot s^{-1}$ at the minimum) to prevent rapid silting. The design flow is derived from the data on the recommended degree ("durability") of protection: 3−5 years for grassland, 5−10 years for arable land, 10−20 years for orchards and vineyards (20−50 years for orchards and vineyards on terraces), 50 years for residential areas, terraces in residential areas, for buildings, structures, etc. in hygienic protection zones and for other specially protected property, and 100 years for important residential and industrial agglomerations. The higher degrees of protection apply primarily to peripheral interception ditches and outlet ditches in which there is no safe flow outside the ditch profile. The flow profile of the interception and outlet ditches successively enlarges with the expansion of the watershed area involved, including also non-agricultural land.

The cross profile of the ditch (Fig. 64) is usually trapezoidal, the gradient of the slopes being 1 : 1.25 to 1 : 1.5; on sandy or clayey soils the gradient may be smaller, 1 : 2−2.5 (PASÁK et al., 1984). Prefabricated ditch-lining panels have unified dimensions, the ditch bottom widths being multiples of 0.3 m. The minimum width used in practice is 0.6 m and the minimum depth is 0.4 m. With the interception ditches the cross profile is extended to include the bank of excavated material and the reduced height of the other bank on the other slope.

The diagram in Fig. 65 (ZUNA, 1987) can be used for designing grass-lined ditches of trapezoidal profile: the diagram applies to ditches with a bottom width of 0.6 m and slope gradients of 1 : 1 and 1 : 1.5. For the given design flow rate and inclination of the ditch bottom, the calculation is based on the determination of the flow module, $K = Q : I^{0.5}$. Using this K parameter, plotted on the x axis, it is possible to identify on the diagram the appropriate water depth y (K_1 applies to a gradient of 1 : 1, $K_{1.5}$ to a gradient of 1 : 1.5) and also the values of T and w. The mean velocity of the flow can be calculated by the equation

$$v = wI^{0.5} \qquad (m \cdot s^{-1}) \tag{5.19}$$

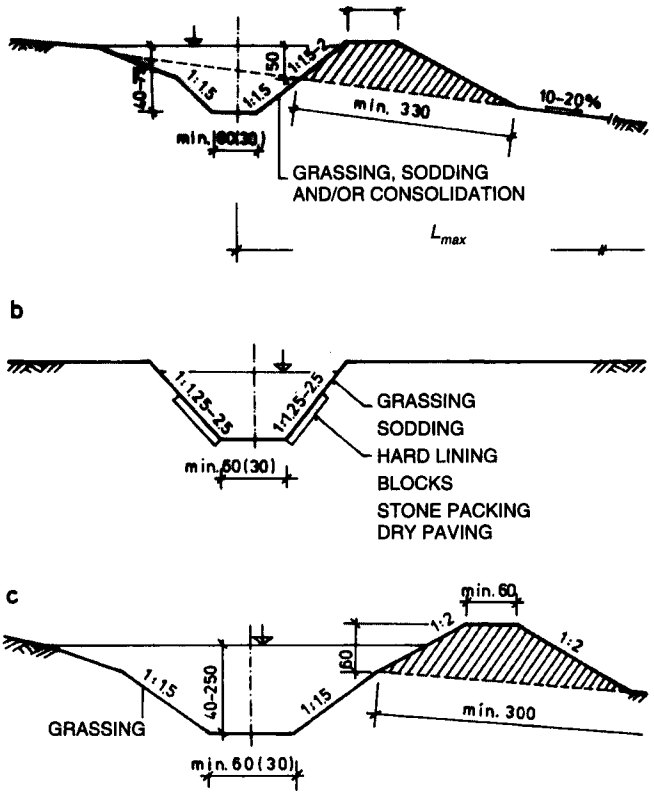


Fig.64 Anti-erosion ditches: a — distribution of collection interception ditches, b — outlet ditch, c — collection ditch.

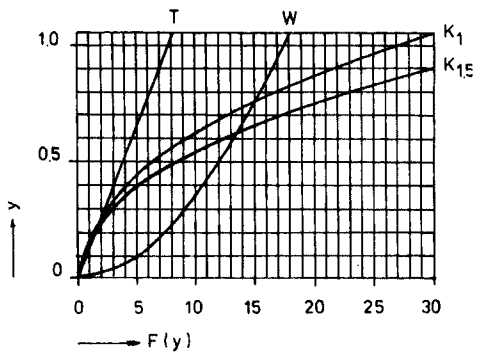


Fig.65 Diagram for the dimensioning of the ditches (after ZUNA, 1987).

and the limit tangential stress τ_m

$$\tau_m = 1000TI \qquad \text{(Pa)}, \tag{5.20}$$

where I – the gradient of the bottom.

For the safe design of a ditch without lining, τ_m should be 80 Pa, otherwise a lining should be provided according to data in Tab. 46.

The networks of channels lining roads can be used with advantage as interception and outlet ditches: field roads and paths break the length of the slope and catch precipitation water not only from the roads themselves but also from the adjacent fields. The best roads for erosion control are those across the slope. The holding capacity of road channels is limited by their depth of 0.4 to 1 m which, in turn, depends on the width of the road and on the traffic safety requirements. It is important that the plane of the road surface should be at least 15 cm above the surface of water in the channel. Depending on the location of the road, the channels may sometimes be used as infiltration ditches for erosion control.

Erosion-control channels that can be crossed by farm machines (Fig. 66) may be built on terraces in orchards and vineyards. They have a "U" profile and are either concreted on site or built of prefab panels. The inner width of such channels is 30 to 50 cm and their height is 30 to 60 cm. Steel grilling covers the top of the channels. When designing such channels, the usual hydrotechnical calculations should be combined with a stress analysis and calculation of the bearing capacity of the ground (the channels should bear the weight of the farm machines crossing

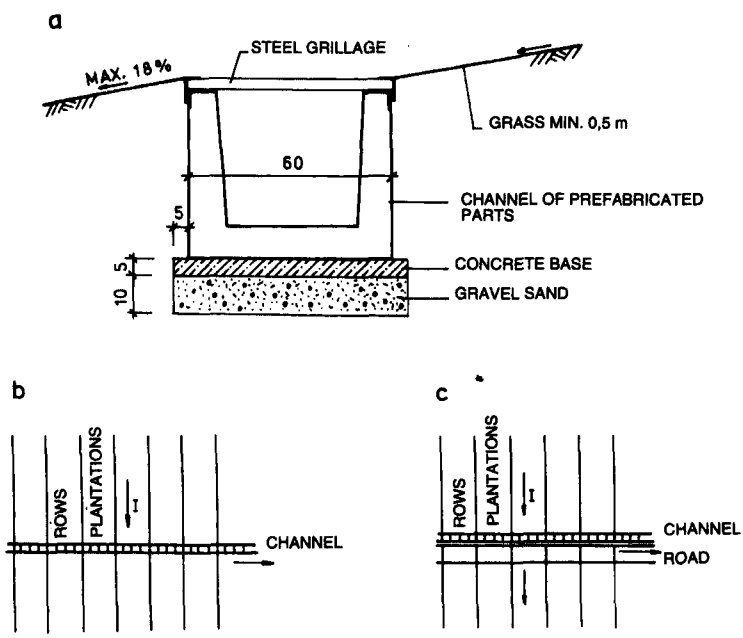


Fig.66 Passable anti-erosion channel: a – cross-section, b – location of the channel within a vineyard or orchard, c – location of the channel for the protection of a road.

them). They are designed either with or without connection with roads or may be built across roads.

Channels and ditches must be regularly cleaned to remove silt and excess vegetation, especially at lower gradients and along roads; the lining must be repaired without unnecessary delay, especially at steeper gradients. Shrubs or trees are planted along the channels and ditches where possible (the vegetation must not obstruct mechanised maintenance). Erosion-control ditches have a conveyance function: they speed up the runoff of water from the landscape. The water should therefore be stored in reservoirs. As the ditches only convey water during heavy rains, they cannot be used to keep aquatic organisms.

5.4.4 Check dams

Check dams have a similar purpose to terrace channels: they catch surface water which then either infiltrates into the soil (if the dams follow the contour line) or is safely conveyed outside the protected area: the latter approach is used where the soils have a heavier texture (hence a poor infiltration potential) or where the rain and snowfalls are too heavy. Detention dams, on the other hand, are built in more or less arid areas and in fields with soils of medium permeability.

Check dams usually cannot be passed over by farm machinery. Their slopes incline at 1 : 1.5 and their base is either low or wide. Narrow check dams 90 − 150 cm wide at base and 15 − 30 cm high are built on permeable soils on slopes of gradients of up to 8%. Wide check dams are safer: 2 − 4 m wide at base and 25 − 45 cm in height (90 cm at the maximum), they are built on steeper slopes and on heavier-textured soils. They need more earth for their construction, but their advantage is that they are not so steep, so that they can be passed by farm machines and put under special crops.

The design of the cross profile should be based on the needed retention space because water should not spill over the dams (Fig. 67). Slopes are usually protected by systems of check dams whose spacing L is calculated by the generally applicable equation for admissible amounts of soil that can be washed away G_{adm} (see Section 5.4.2). The retention space must meet the condition that

$$h/2[h/I + hn] = \varphi i_s t_d L\,, \tag{5.21}$$

where h − height of the check dam (m),
n − gradient of the dam's slopes (1 : n),
I − gradient of the hillside slope,
i_S − average intensity of design rain ($m \cdot s^{-1}$),

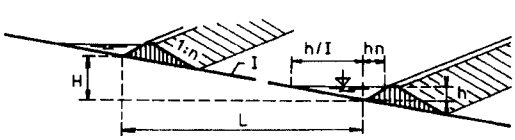


Fig.67 Check dams.

t_d – duration of the design rain (s),
φ – runoff coefficient,
L – spacing of the dams (m).

The determination of the length D of the inclined dams should meet the requirement that most of the water should flow away during the course of the rain, hence,

$$D = vt_d \quad \text{(m)}, \tag{5.22}$$

where v – the velocity of flow of water along the dam, calculated from the Chézy equation ($m \cdot s^{-1}$),
t_d – duration of the design rain (s).

Temporary dams are a special type used in areas with poor precipitation to catch all the surface runoff during the period outside the growing season; at the same time, temporary dams eliminate the erosion damage caused by snowmelt. Temporary dams are made in autumn using a special plough on hillsides of gradients of 3–12%; they are about 50 cm high and 12 to 50 m apart, depending on the gradient of slope. They more or less follow the contour lines. To prevent water from running along the dam, the dams are arched to make loops on the upgrade side (Fig. 68).

Another temporary measure to check erosion is the placing of strong plastic foils vertically in the soil to force water to infiltrate into the soil. Their installation must follow the contour line. Good experience with the use of this method has been acquired on soils of light to medium texture under root crops and maize. They can be passed easily because they are flexible; on the other hand, they are difficult to remove and remnants thereof may be scattered over the field and pollute the soil.

Check dams built to control erosion, especially those impassable for farm machines, often hinder cultivation: fields where they are built must be cultivated along the contours for which special machinery is needed and the dams themselves take up arable land. On the other hand, they are comparatively cheap (SEDLÁK, 1976). Sets of parallel dams are therefore built everywhere possible and the ground has to be levelled for this purpose in some places. Special ploughs or graders are used to make the dams and the earth needed to build them is either taken from the shallow channels on both sides of the dam or carried to the spot from other places. A set of check dams in which the gradient of slope between the crest of one dam and the toe of the next is substantially reduced may be a first step in terracing. The dams designed to convey excess water lead to grass-covered terrace channels, ditches, etc.

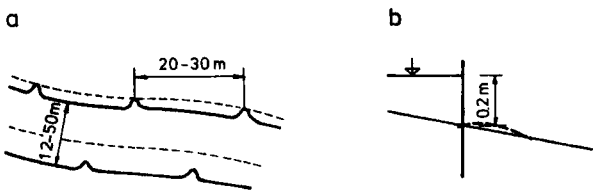


Fig.68 Temporary check dams (a) and interception plastic film (b).

Systems of detention dams are used in the countries of the former USSR, India, Java and in other parts of the world where terraces are built for tea, gum trees and other crop plantations (JŮVA and CABLÍK, 1954).

5.4.5 Terraces

Terraces reduce the gradient of slopes of hillside fields and break the long surface runoff paths that would cause erosion; the land on terraces can be cultivated by large-scale methods. Terraces are costly, long standing and extensive interventions in the landscape which may sometimes have adverse environmental effects; hence, all effects on water management, soil, farming operations, environment and landscape, wildlife and conservation of cultural riches, as well as geology, economy, energy saving, etc. must have been considered before terraces are designed.

Earth terraces are designed for erosion control on hillsides with slope gradients of 15 to 30% or more and with a medium to high hazard of erosion. It is important that the terrace area should allow for profitable crop specialisation: for orchards and vineyards the area should be 50 – 100 ha at the minimum, in irrigated terraces it should be at least 200 ha. Terraces are built with advantage on the southern, southeastern and southwestern slopes with less danger of frost damage, with lower rates of occurrence of pests and with much better soil-water regimes (higher infiltration) on the terraces.

The shape and height of the terraces depend on the gradient of the slope, on the depth of soil profile, on the volume of earthwork needed, on the accessibility of the terraces to transport, on the crop to be grown, etc. Terrace features are described in Fig. 69.

Terrace design must rely on geological survey. Loess and other compact earths make the best ground on which to build terraces; clayey, gravelly and sandy soils are less suitable and a stony or rock ground is unsuitable. It is not recommended to build terraces on hillsides exposed to the danger of landsliding, in large water-

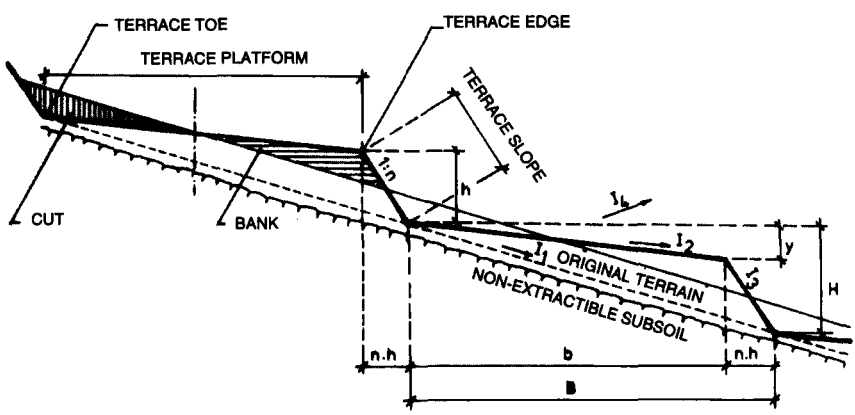


Fig.69 Diagram of an earthen terrace.

logged areas or on hillsides where the soils are shallow. Preliminary geological study must be followed by detailed geological research using boreholes 3 m deep for earthworks (cut or embankment) up to 2.5 m; if the cut is to be deeper or the embankment higher than 2.5 m, the holes must be deeper, hence more costly. The number of the holes depends on geological conditions. Soil research is another important step which provides technical data such as soil depth and the pedological characteristics and also data concerning the soil's biological characteristics and information needed for the reclamation of the soils on the terraces.

The terraced area must be protected against both surface and ground water (this also applies to the period of construction) and against internal water whose volume is measured as shown in Section 3.1.2. Isolated waterlogged places (springs) and small areas of land prone to sliding inside the terraced territory should be drained and stabilised (BENETIN et al., 1987). In the protective zones along (or around) the surface sources of drinking water, all the soil displaced by erosion must be trapped in settling pits or erosion-control reservoirs.

On terraces 3 to 6 m high, the slopes are recommended to be inclined at 1 : 1.25 to 1 : 2, depending on the compactness (loess, other cohesive earths) or looseness (fine sand) of the earth. Terraces higher than 6 m should be assessed for stability, which is to be 1.3 at the minimum. Slopes higher than 8 m should be interrupted by a berm at least 3 m wide. Knowledge of the stability data is extremely important above settlements, farm or industrial facilities, roads, underground pipelines or cables, water reservoirs and streams, etc.

In designing terraces, heed should be taken of the general appearance of the landscape, especially in places offering attractive views and in protected areas. Protection is also required for archaeological sites, protected greenery, architectural and cultural monuments and landscape sites. Ecologically, terraces are an intense intervention in the natural relationships within (and between) plant, animal and microbiological associations; at least parts of the original terrain have to be saved with their natural plant communities (trees and shrubs for nesting, breeding grounds for butterflies, etc.) which would provide shelter for many animal species. Also included in the terrace projects must be biological corridors to save or extend the environmental stability of the landscape, associated with the biological plan for the landscape with indication of biological centres.

Some parts of the ecosystem will need special protection against the possible adverse effects of farming on the terraces (use of chemicals, commercial fertilisers, removal of soil by erosion, mechanical damage).

There are either narrow or wide terraces or terrace plots (Figs. 70, 71, 72).

Narrow terraces are usually 3−5 m in width and follow the contours of the original terrain: hence, in terrains of uneven slope gradients their widths vary and additional terraces must be built between the contour benches where necessary. Such terraces usually bear only one, exceptionally two, rows of fruit trees or grapevines which are planted along the edge of the step to allow farm machines to drive along the bank of the next step (or between the rows in the two-row terraces). Though narrow terraces are fairly inexpensive to build, they do not effectively recompense their costs (as shown by the example of the mass terracing projects of the 1960's in the Czech Republic). Today they are sporadically built on shallow to medium deep soils.

Fig.70 Terraced slopes (Photo J. Sanetrník).

Fig.71 Narrow terraces (Photo J. Sanetrník).

For the wide terraces the minimum width is 8 – 12 m (depending on row spacing) for vineyards, 10 – 20 m for orchards and 20 m for arable fields. They may be subdivided into medium wide (up to 50 m) and very wide (above 50 m) terraces. Widths from 40 to 60 m allow modern large-scale cultivation of permanent crops.

Terrace plots are pieces of land with a reduced gradient of slope whose

Fig.72 Terrace construction (Photo J. Filip).

Fig.73 Biological pest control in orchards on terraces (Photo F. Toman).

minimum width is 20 m; their "length" does not necessarily need to be greater than their width. Terrace plots are usually separated from the land around not only by the upper and lower steps but also by a steep bench on the side(s) (Fig. 74).

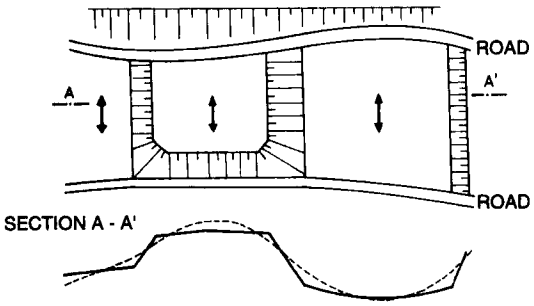


Fig.74 Terrace plot.

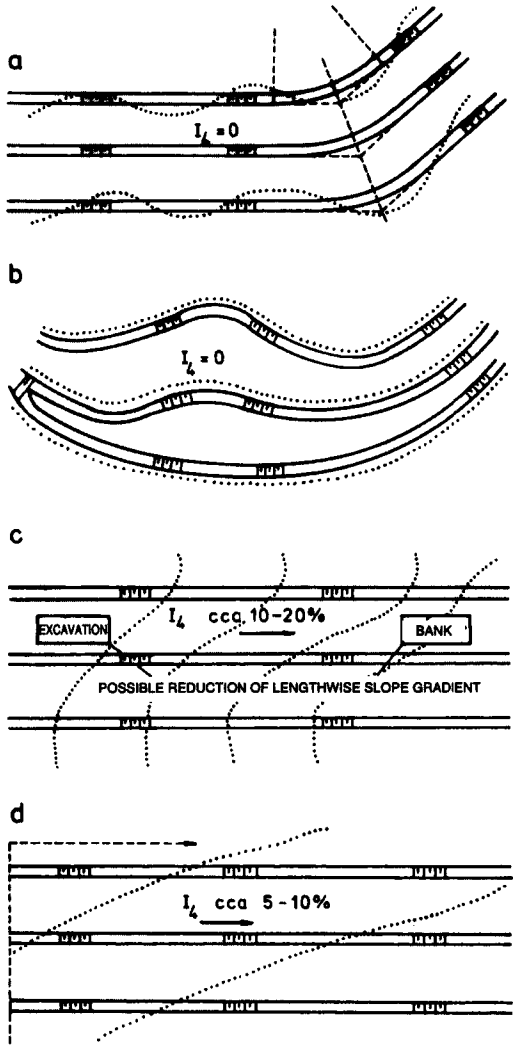


Fig.75 Terraces: a – near-contour,
b – contour,
c – sloping,
d – intermediate.

Experience gained in designing and building terraces, combined with experimental results, has led to the development of the following terrace construction types: near-contour, contour, sloping and intermediate (Fig. 75). Their main characteristics are shown in Tab. 47. Projects for irrigated terraces are being prepared for construction in the dry southern areas of Moravia (Czech Republic) and Slovakia: in such projects account should be taken of the distribution of the irrigation pipeline network (the pipes should be located in the natural parts of the terrain), the width of the terraces should be adjusted to the technical requirements of irrigation and the operations of terracing and irrigation-system construction should be arranged into an optimum sequence.

The length of the terraces depends on the topography, on the accessibility of the terrain to machinery and on the cultivation operations and methods. For example, woody fruit species have to be planted in straight rows. A minimum length of 80 m is recommended for vineyards and orchards; the optimum length is 150 m (with access roads on both sides, 300 m). For arable fields the optimum length ranges from 200 to 500 m with access from both sides. Each terrace must be connected with a road which should be lined by trees or shrubs (for environmental reasons, for the farm workers to have a rest in the shade, etc.). Changes in the lengthwise direction may be either sharp (angular), suitable for vineyards, or arched, which look more shapely.

The cross slope of the terrace flat (positive slope, inclined towards the terrace edge) should preferably range from 3 to 5% (7% at the maximum) so as not to allow too much soil to be removed by erosion. Sometimes, there may be a zero inclination or the surface may slope down towards the toe of the next (upper) terrace to catch and infiltrate precipitation water; however, such solutions need more earthwork and involve a greater risk of landslide.

Table 47

Characteristics of terrace types

Terraces	Longitudinal direction	Longitudinal gradient	Widths of terrace flats	Excavated earth shifted	Relationship between terraces
Near countour	more or less following the contour lines	3%	Usually the same	lengthwise	parallel
Contour	along contour lines	0%	Unequal	crosswise	nonparallel
Sloping	not respecting the contour lines – along the slope	10–20%	Usually the same	usually lengthwise	parallel
Intermediate	oblique to contour lines	5–10%	Usually the same	usually lengthwise	parallel

The terrace banks are stabilised either by vegetation (the so-called earthen bench terraces) or by retaining walls (dry rubble masonry, cyclopean concrete, cast or precast iron concrete). Terraces with retaining walls can reduce the loss of productive area by as much as 90% and are suitable for very steep slopes (30−35%). Any project of this kind must be subjected to stress analysis. The maintenance costs of the retaining walls are low, but the costs of building them are always high, so that such projects are exceptional.

Terrace projects must also include roads. Classified as "special purpose roads in inclined terrains" they can be designed for exceptionally low speeds ($10\ km \cdot h^{-1}$) with a short radius of curve (15 m) and high gradients (12−15%). Access to the terraces is provided by main roads in the middle of the terraced area or by peripheral roads. These are usually one-lane roads with passing lay-bys 4 m wide at the crest. In large terraced areas, access to the terraces is provided by two-lane main roads 6 m wide at the crest. Subsidiary roads connect the main ones with the terraces. The width of such roads is 4 m of which 3 m is rigid and the construction of their end part will depend on whether the terrace is at the same level as the road, or above or under it. Crash barriers are required if the road embankment is higher than 3 m. The project of the roads must also include the road channels with their draining function (see Section 5.4.3).

Headlands (for the machines to turn at the end of the terrace) are designed as separate grounds usually 10 m wide (12 m for combine harvesters). Roads will often well serve this purpose. Steps (1 m wide, cut in the ground or made of stone or concrete) are sometimes built for pedestrians to get to the terraces.

Terrace projects must also include solutions for the changed hydrographic conditions: there should be erosion-control ditches, a drain system and other erosion-control features.

Construction of terraces involves extensive earthworks, using dozers, scrapers, sometimes excavators, trucks and other machines. The building operations must be well organised. The verdure that is to be left in place should have been marked before the construction work begins and must be well protected during the whole construction period. The building site must also be protected against both surface and ground water, usually by a peripheral interception or temporary ditch.

It is important to prepare the sequence of operations in building the terraces thoroughly. To protect the site against surface water, the work usually starts from the uppermost terrace. The valuable topsoil has to be saved, the thickness of the stripped topsoil layer depending on quality: it should contain at least 2% humus − an amount usually contained in the upper 20 cm, though on the valley floor such a rich layer may be much thicker, sometimes even 200 cm. The flowing system of construction can be used with advantage: the topsoil from the first terrace is deposited outside the site (after a year it must be turned) and the finished first terrace is then covered with topsoil from the next; the topsoil from the first terrace is then put onto the last. The earth volumes excavated and filled should be more or less balanced on each terrace, the earth being shifted both lengthwise and crosswise. Slide control

cuts have to be made in the ground to provide stability of high embankments, especially on the bottoms of valleys and in vast horizontal shifts. Additional drainage has to be provided if water-bearing horizons are encountered, especially on the boundaries of Quaternary and Tertiary sediments. Terrace embankments are built by spreading the earth in 20 cm layers, using a bulldozer. If the standard PROCTOR test says that a higher degree of compactness is needed, additional compacting or rolling has to be provided.

Flood rains during terrace construction may bring considerable erosion, sometimes there is even a risk of landslide. Hence, as soon as the terrace slopes are finished they must be covered with a 5 – 10 cm layer of humus (thicker layers would not allow all roots to penetrate under the humus) and should be sown or planted with woody species. The finished terrace flats should immediately be ploughed (before the topsoil is spread). If the potentially cultivable subsoil layer is thick enough (30 cm at the minimum) the thickness of topsoil to be spread over a terrace to be used as arable land should be 30 cm at the minimum; for orchards and vineyards both the topsoil and subsoil should be 40 m thick. For subsequent biological reclamation, it must be possible to cultivate (by tillage or trenching) a layer of at least 60 cm for arable crops and about 80 cm for orchards and vineyards. Thicker topsoil layers are better because they make the reclamation easier and cheaper.

Reclamation is the final stage of terracing, determining the success of the effort, i.e., the yields of the crops to be grown. A number of factors are involved, including the degree of soil degradation, the quality of the topsoil spread on the terrace surface, the natural conditions and the general economic and management conditions. The process of reclamation lasts 3 – 5 years and its purpose is to restore the soil to biological life. The soil is subjected to trenching (deep loosening), store dressing (addition of phosphorus, potassium and magnesium at rates of 3 – 4 t per ha), reclamation sweetening to adjust soil acidity, organic manuring or addition of compost (80 – 100 tonnes per ha) to provide the required microorganisms; green manuring is followed by the cultivation of soil-improving crops such as clover (PASÁK et al., 1984).

5.4.6 Erosion-control reservoirs

Erosion-control reservoirs or, more frequently, ponds and multiple-purpose reservoirs with a prevailing anti-erosion function, represent an important – though rather complex and costly – erosion-control measure. Their purpose is to accumulate, retain and retard surface runoff and settle the eroded material, thus protecting the downstream fields, residential areas and farm and industrial facilities against floods and sediments. They are usually built in areas where the soil is heavily eroded and large amounts of material are carried away by water.

Depending on their purpose, erosion-control reservoirs can be divided into protective, interception, settling, seepage and sanitation reservoirs or ponds (ŠÁLEK et al., 1983).

Protective or detention reservoirs are built on water courses to safely check flood waves which would otherwise cause damage to downstream fields,

buildings, roads, etc. If the detained water is more or less clean, such reservoirs simply have a flood-control function. If the reservoir catches the sediments brought by the water it also has a detention function. Small erosion-control reservoirs are very effective on mountain torrents of small catchment areas where abrupt and high flood waves occur which, however, are of short duration so that they can be stored in small reservoirs. For most of the time the storage space of such reservoirs is empty so as to be able to check flood waves and then to successively discharge the water. Down in the valleys where flood waves combine from many triburaties, the floods last longer and have high peak flow rates; in such situations the reservoirs must have a large storage capacity and serve several purposes as structures of water-management importance (KRATOCHVÍL, 1961).

Correct functioning of the interception reservoirs depends on their location, construction, operation and maintenance. As far as possible, they should be located on poorer land, in gravel pits, etc. They are usually designed as temporary flow-through reservoirs which, after silting, are reclaimed into fields, meadows or woodland; they may also be designed as permanent reservoirs with regular cleaning. In such cases, the project must include solutions for extracting the sediment and there must be a road along which the extracted material can be carried to the place of processing or use. Interception reservoirs may either contain some water (less than capacity) or may be empty and dry to be filled only when there is a flood; while they are empty, such reservoirs serve as meadows. The soil near the dam may often become waterlogged, so drainage is sometimes needed.

The storage capacity A_r of the reservoir which is, in the case of the dry reservoirs, the whole controlled storage, equals the volume W_k of the flood wave above the maximum safe flow rate Q_k (flow not causing damage). However, the volume W_k above the limit flow Q_k varies with floods, hence, there is only a limited probability that the flow rate Q_k will not be exceeded and this probability is referred to as the degree of protection for the respective n-year flow. Erosion-control reservoirs are designed on the basis of an assessment of the damage caused by floods and are to protect the downstream areas against floods calculated as a 50- to 100-year average of the flow portions above the safe limit Q_k. It must also be taken into account what changes, conducive to higher maximum flow rates, can be expected to be caused by human activity in the watershed (VITHA, 1984). Hence, the flood flows in the given water course must have been subjected to thorough hydrological analysis before the reservoir's storage volume is designed (VOTRUBA and BROŽA, 1980).

For preliminary orientation, the volume of water in the reservoir (ŠÁLEK et al., 1983) can be computed by the formula

$$V_A = h_{\max} \frac{S}{3} \quad (\mathrm{m}^3), \tag{5.23}$$

where S – the surface area of water in the reservoir (m^2),
$h_{\max}$ – the maximum depth near the dam (m).

The approximate value of the volume of the dam can be calculated by the formula

$$V_H = \frac{L_h H}{2}\left(b + \frac{1}{n} + \frac{1}{m}\right)\frac{H}{3} \qquad (\mathrm{m}^3) \tag{5.24}$$

where L_h – length of the dam (m),
H – average height of the dam (m),
b – width of the dam's crest (m),
n – gradient of the upstream side slope (1 : n),
m – gradient of the downstream side slope (1 : m).

The flood flow is also reduced by the reservoir's uncontrolled storage A_n which exists in all reservoirs. The magnitude of the reduction (transformation) of the flood flow depends on the storage volume A. Neglect of its value will lead to overdimensioning of the safety spillway and of the structures further downstream: this generally increases the costs. In small reservoirs the reduction Q_s of the flood flow can be approximated using the equation developed by JŮVA et al. (1980):

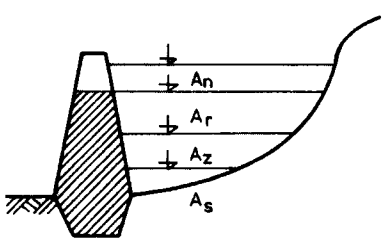
$$Q_s = Q_{\max} - \frac{2A_n}{t_p} \qquad (\mathrm{m}^3 \cdot \mathrm{s}^{-1}), \tag{5.25}$$

where $Q_{\max}$ – the respective flood flow ($\mathrm{m}^3 \cdot \mathrm{s}^{-1}$),
t_p – time (duration) of the flood flow (s).

The calculation of the storage volume of the interception reservoir must take account of filling with sediment.

The reservoir dam must retain water safely: if it breaks, the consequences may be disastrous. Design principles are to be found in the specialised technical

Fig.76 Reservoir storage: A_n – surcharge storage,
A_r – controllable storage,
A_z – reserve storage,
A_s – permanent storage.



literature. The dam must be stable and should not leak. The dam's crest should be at least 60 cm above the maximum water level and its width (3 m at the minimum) is calculated from the stress analysis (static design) of the dam. The dam should have an outlet gate and the reservoir must be protected against floods by a safety spill.

Torrent dam systems are built on mountain torrents to serve the same purpose as the interception reservoirs.

Scrapers, bulldozers, excavators, dumpers and other trucks, compacting machines, concreting equipment and equipment to handle reinforcing bars, etc., all these find use in the construction of reservoirs. The prescribed technology must be adhered to during all earthworking and building operations and particular attention should be paid to the place of contact of the structure with the groundwork. The finished dam must be regularly inspected, maintained and repaired; permanent erosion-control reservoirs must be regularly cleaned and the extracted sediment can be used – depending on its composition – for composting, for road pavement, etc.

Settling reservoirs serve for trapping the eroded material from smaller areas exposed to erosion to prevent the soil from being transported far downstream. They have the shape of widened rectangular channels in which the water carrying eroded material loses its speed and "dumps" the material on the bottom. The time of incidence t_p of the particles onto the bottom depends on water depth h (m) and sedimentation rate w ($m \cdot s^{-1}$), hence,

$$t_p = h/w \qquad \text{(s)}. \tag{5.26}$$

Then the length of the settling reservoir

$$L = vt_p \qquad \text{(m)}, \tag{5.27}$$

where v – average velocity at which water moves through the reservoir ($m \cdot s^{-1}$).

A similar method of calculating and designing settling reservoirs is described by LACO (1976).

The sites for settling reservoirs are selected so that at least two reservoirs can be built next to each other; they must be easily accessible for the machines needed for extracting and removing the sediment.

The seepage-type erosion-control reservoirs are to check not only the eroded material (mainly topsoil) but also the surface water which, as a result of seepage, becomes groundwater. Like the settling reservoirs, the seepage reservoirs have the shape of rectangular channels but their bottoms must be permeable. Collecting ditches which intercept the surface flow empty into the reservoirs, so each reservoir must also have a safety outlet ditch. The sediment is regularly extracted and composted. In some countries seepage reservoirs are used in vineyards (ŠÁLEK et al., 1983).

The so-called limans are a special type of seepage reservoirs. They are designed to intercept snowmelt waters in spring (Fig. 77). Limans are small earth dams 0.7 to 2 m high and 500–600 m long which detain water by spreading it to a width of 80–150 m (40 m at the minimum); the average depth of water in these "lakes" is 0.5–1 m. The water that fails to infiltrate into the ground in 5 to 15 days is successively discharged through controlled outlet gates. Safety spillways prevent water from running over the dams. It is advantageous to build limans as steps ("storey limans"). In the arid regions of the Ukraine, along the Volga river and other areas, limans are used to provide a reserve of moisture for the growing season. During the period when they are empty, limans can be managed as meadows, pastures, woodland, orchards or arable fields; however, they are not suitable for large-scale operations.

Sanitary reservoirs are special interception reservoirs, serving for the stabilisation of deep gullies. For the most part, they have a small storage capacity. Their dams are usually built of stone or concrete and have no outlet gate. When they are filled with sediment the original gradient of the bottom is reduced and the crest of the dam forms a new (higher) level of erosion base.

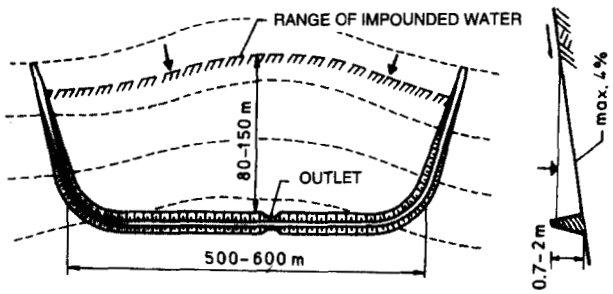


Fig.77 Liman check.

CHAPTER 6

Torrent control

6.1 Torrent characteristics

The torrent is a water-course category defined as a stream *with highly variable discharges, high slope gradients of the bottom, high scouring activity, transport and deposition of sediment and frequent changes of channel dimensions, the main criterion being the formation, transport and deposition of sediment.*

The variation of the discharges, i.e., the ratio between the minimum and maximum discharges, may be as wide as 1 : 5000 or even wider (RIEDL and ZACHAR, 1973). Abrupt changes in torrent discharges usually occur during flood rains following a long wet period when the soil in the watershed no longer has sufficient capacity to absorb the flood rain water.

It is a characteristic feature of torrents that their discharge grows rapidly to reach a maximum and subsequenty drops again equally rapidly. Torrents have small watersheds, so high rainfalls usually affect the whole watershed area. With the high inclination of the ground, the surface runoff rapidly concentrates in the channel and can reach the lower segments of the torrent during the course of the rainfall.

The key criterion of the "torrent" nature of the stream is its scouring activity by which gravel sediment is released and transported downstream. The sources of sediment include the torrent channel itself and the deposits of gravel carried thereto from the steep slopes of the valleys exposed to erosion and from tributary ravines.

The scouring and transport of sediment from the torrent channel are due to its high slope and the lack of balance in flow directions. The higher the channel gradient, the greater the carrying power of the water stream which shifts gravel benches, releases further amounts of sediment and deepens the bottom (Fig. 78). In torrent channels with nonregulated flow direction, the concave banks are exposed to a high pressure conducive to extensive bank scourings and to loosening of large amounts of gravel. Water carries the sediment, raised from the torrent channel, farther downstream and deposits it again in places where the torrent has a lower slope, and hence a lower carrying power (Fig. 79). This causes further damage because the gravel silts up both the torrent channel and the adjacent fields (Fig. 80).

The definition given in the first paragraph of this chapter is too broad, expressing the properties of torrential water courses too generally so that it does not provide a sufficient basis for deciding whether to include a particular water course in the torrent category. To remedy this, a number of classification methods have been developed, based on specifications of the extent of the erosion processes and factors that modify them.

Fig.78 Deepened channel of the regulated Červená torrent in the Ore Mts., Bohemia (Photo V. Škopek).

Fig.79 Sediment deposits in a torrent channel (Photo V. Škopek).

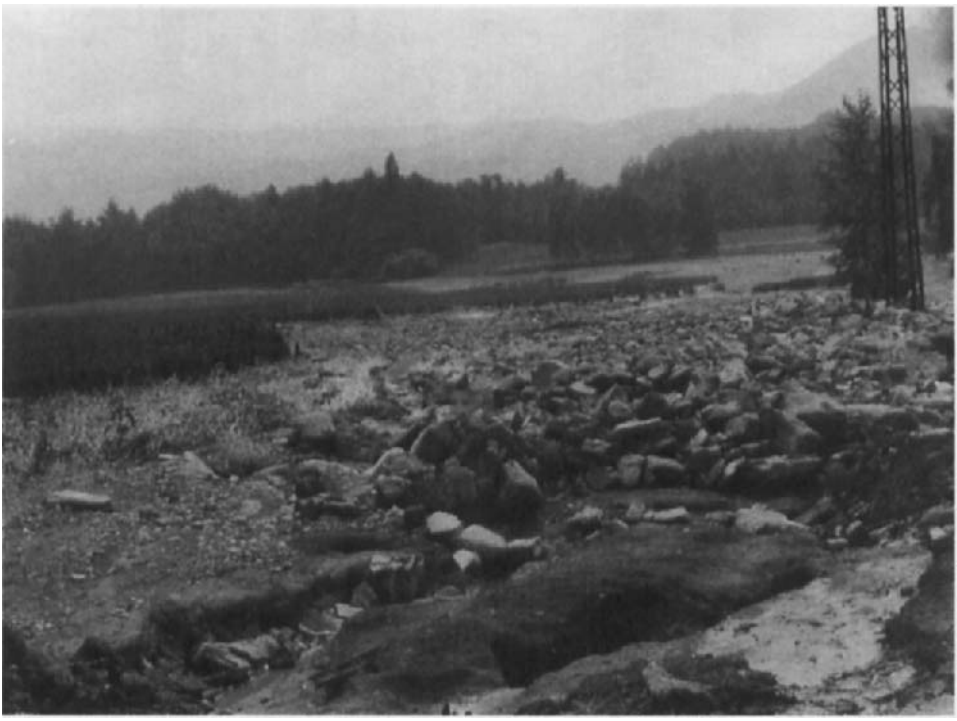


Fig.80 Sediment deposits in a cereal stand after a flood on the Tisá torrent in the Ore Mts., Bohemia (Photo P. Vrzák).

ŠKOPEK (1982, 1987) used a special coefficient to distinguish torrents from other streams and rivers. This "torrent coefficient" K_b of the watershed is derived from a modified version of the *Gavrilovich* (1972) *formula*:

$$K_b = \frac{HOV_S PE(S+1)^{1/2}}{L(S_Z+1)^{1/2}}, \tag{6.1}$$

where H – density of hydrographic network ($km \cdot km^{-2}$),
O – length of the watershed line (km),
V_S – mean altitude difference,
P – coefficient of the mean permeability of the soils,
E – coefficient of the watershed's propensity for erosion,
S – watershed area (km^2),
L – length of the main stream (km),
S_Z – area of the afforested part of the watershed (km^2).

The density of the hydrographic network can be calculated from the formula:

$$H = \frac{L + \sum L_i}{S} \quad (km \cdot km^{-2}), \tag{6.2}$$

where L_i – lengths of the separate tributaries (km).

The mean altitude difference of the watershed is determined from the formula:

$$V_S = V_p - V_u \quad \text{(km)}, \tag{6.3}$$

where V_p – the watershed's average altitude above sea level,
V_u – altitude of the mouth of the stream or of the profile for which the calculation is made (km).

The average altitude of the watershed is calculated by the following equation:

$$V_p = \frac{\sum S_i h_i}{S} \quad \text{(km)}, \tag{6.4}$$

where S_i – area of the part of watershed between two neighbouring contour lines (km^2),
h_i – mean altitude between two neighbouring contour lines (km).

Table 48

Coefficient of mean soil permeability

P	Degree of soil permeability	Type of soil
1.0	totally impervious	heavy clayey soil rocks
0.9	impervious	clayey soil
0.8		peat, swamps
0.7	not very permeable	clay loam grey forest podzol
0.65		loam to clay loam
0.6	permeable	loamy soil limy chernozem
0.55		loamy sand
0.5		loamy sand to sandy loam
0.45	very permeable	sandy soil, sands, gravels

The mean permeability coefficients for the different types of soil are shown in Tab. 48. The coefficient of the watershed's propensity for erosion is shown in Tab. 49.

In the case of sparsely or sporadically afforested watersheds, a substitute value (S_Z) of the soil's vegetation cover will be used instead of the data on the area of the afforested part of the watershed. This substitute figure is calculated as follows:

$$S_Z = 0.6s_l + 0.8s_t + 1.0s_o \quad \text{(km}^2\text{)}, \tag{6.5}$$

Table 49

Coefficient of a watershed's propensity for erosion

E	Intensity of erosion in the watershed and stream channel
1.0	Whole watershed affected by all types of erosion, stream channel devastated by both lengthwise and crosswise erosion, rough sediment continuously removed, transported and deposited. Exposed soil surface without sufficient vegetation cover prevails in the whole watershed. The slopes have a gradient of more than 50%.
0.9	Up to 80% of the watershed area is affected by rill and gully erosion. Transport and accumulation of rough sediment prevail in the stream channel.
0.8	Up to 50% of the area of the watershed is affected by furrow, rill and gully erosion. The gradient of the slopes is above 30%. There is significant transport and intensive accumulation of coarse sediment in the stream channel.
0.7	Furrow and rill erosion types prevail in the watershed. The slope gradient is above 20%. Gravel and cobbles are transported in the stream channel.
0.6	Sheet erosion and sporadically also rill erosion, prevail in the watershed. There is significant crosswise and lengthwise erosion in the channel, with transport of gravel.
0.5	Sheet erosion affects up to 50% of the watershed, furrow erosion becomes rill erosion, gravel is transported and accumulated in the channel. The slopes in the watershed have gradients of up to 20%.
0.4	25 – 30% of the watershed area is affected by sheet erosion, furrow erosion occurs in some places. There are sites of movement of finer sediment – this sediment is transported and deposited in the stream channel. The gradient of the slopes is 10 – 15%. The vegetation cover is disturbed – forests are affected by industrial emissions.
0.3	About 20% of the watershed area is affected by sheet erosion, in some places by furrow erosion. There are distinct signs of the topsoil's being washed away. Fine sediment is transported in the channel of the stream.
0.2	The whole watershed is free of distinct signs of erosion. There is a large proportion of farmed land in the watershed. Sediment largely develops through erosion in the stream channel. The slopes have gradients of up to 20%.
0.1 – 0	Whole watershed free of visible signs of erosion. Forest covers a prevailing part of the area and has a good species and age structure. The remaining area is perennial grassland. The channel of the water course is stabilised in both direction and gradient.

where s_l – afforested part of watershed (km^2),
s_t – part of watershed covered by permanent grassland (meadows, pasture) (km^2),
s_o – part of watershed with predominance of arable land or bare soil surface (vineyards, hop gardens) (km^2).

Table 50

Categories of torrents and other water courses according to the torrent coefficient

Category	Torrent coefficient K_b	Characteristics of the water course
I	< 0.1	water course of non-torrential nature
II	0.1 – 0.4	torrent with a low intensity of erosion
III	0.4 – 0.7	torrent with medium strong torrent characteristics
IV	0.7 – 1.0	torrent with strong torrent characteristics
V	> 1.0	torrent with very strong torrent characteristics

The torrent coefficient, calculated from equation (6.1), is the criterion by which a particular water course is, or is not, included in the torrent category (Tab. 50).

6.2 Sediments

The torrent bed consists of a mixture of sand, gravel, cobbles and boulders, generally referred to as sediment. The mixture itself consists of particles (grains) whose properties depend on their geological origin. The majority of the grains, determining the general nature of the whole mixture, originate from the same rock. In addition to these, the sediment of the majority of torrents also contains a smaller proportion of grains of other kinds of rock. Downstream of confluences with tributaries carrying debris of a different geological origin, some torrents' sediment may consist of a heterogeneous mixture, containing different proportions of grains of different rocks, characterised by different properties.

The sediment is an important category in torrents. Its composition, including grains of different sizes and shapes, influences the micro- and macrorelief of the torrent bed surface which, in turn, influences the characteristics of the flow. The presence of larger grain categories (gravel, pebbles, boulders) controls the morphology of the surface layer of the torrent bottom, thus influencing the turbulence of the flow. The shape of the grains controls mainly the mode of deposition of the different parts of sediment in the surface layer of the bottom, thus determining the resistance of separate grains and whole bed-load sediments to being lifted and transported.

6.2.1 Properties of sediment grains

6.2.1.1 Sediment grain size

Grain size is determined by three key dimensions, *a*, *b*, *c*, perpendicular to each other, *a* being the length of the longest axis, *b* the length of the medium axis and *c* the length of the shortest axis. The medium axis *b* is the most important of all, referred to as the mean grain dimension *d*.

Sieves are used to determine the mean grain dimension in fine-grained sediment materials. Coarser grains (diameters above 0.08 m) can be measured by passing them through ring gauges or by using a large slide caliber to measure axis *b*.

6.2.1.2 Sediment grain density and mass

The density of the sediment depends on its geological origin. Though the density of each mineral varies with place of occurrence, average density data for different rocks can be used with advantage (Tab. 51).

Density is the key factor controlling the weight of the sediment grains. The other factors on which density depends are grain size and shape.

The mass of small sediment grains of the bed load can be measured by weighing. However, the sediment mixtures of torrents also include large boulders which cannot be picked from the bottom to be weighed. The mass of such large "grains" can be calculated from the dependence of mass on the mean dimension d (VALTÝNI, 1967):

$$G = ad^b \quad \text{(kg)}, \tag{6.6}$$

where a, b – coefficients expressing the effect of grain shape. Data on these coefficients are shown in Tab. 52 (NOVÁK, 1987).

The mean dimension d (axis b) can be measured in almost all large boulders. If it is possible to measure all the axes of the large sediment grains, the mass can be calculated from the formula:

$$G = V\varrho_m \quad \text{(kg)}, \tag{6.7}$$

where V – grain volume (m^3),
ϱ_m – sediment density ($kg \cdot m^{-3}$).

Grain volume is determined from the equation:

$$V = \alpha abc \quad (m^3), \tag{6.8}$$

where α – correction coefficient,
a, b, c – axes of the sediment grain.

The values of the correction coefficient for different sediment grain shapes are shown in Tab. 53 (NOVÁK, 1987).

Table 51

Average density values ϱ_m of different rocks

Rock	ϱ_m $kg \cdot m^{-3}$	Rock	ϱ_m $kg \cdot m^{-3}$
Granite	2620	Mica schist	2710
Silica porphyry	2580	Phyllite	2700
Migmatite	2690	Sandstones, perm	2550
Gneiss	2630	Sandstones, flysh	2550
Granodiorite	2700	Sandstones, chalk	2000
Granulite	2630	Greywacke	2680
Basalt	2860	Marlite	2470
Slates	2680		

Table 52

Values of coefficients a and b for different rocks

Rock	a	b	Rock	a	b
Granite	0.0011	3.007	Mica schist	0.0043	2.336
Silica porphyry	0.0022	2.628	Phyllite	0.0022	2.506
Migmatite	0.0016	2.691	Sandstones, permocarb.	0.0013	2.830
Gneiss	0.0028	2.444	Sandstones, flysh	0.0013	2.806
Granodiorite	0.0013	2.847	Sandstones, chalk	0.0014	2.736
Granulite	0.0015	2.685	Marlites	0.0013	2.680
Basalt	0.0016	2.783	Greywacke	0.0011	2.847
Slate	0.0015	2.650			

Table 53

Correction coefficient α for the different shape categories of grains

Shape category	α	Shape category	α
Sphere	0.41	Disk	0.46
Cube	0.43	Plate	0.48
Ellipsoid	0.43	Rod	0.53
Prism	0.44		

6.2.1.3 Sediment grain shape

Shape is among the key properties of sediment grains. The formation of the surface layer of the bottom depends on sediment grain shape as also does, in particular, the resistance the flow has to overcome while passing by the sediment grains; in this way the grain's shape determines whether the grain will or will not be raised from the bottom and transported by the flow.

The shape of the sediment grains can be expressed by the shape coefficients; the most important of these for expressing the hydraulic characteristics of the bed-load material are:

- coefficient of grain flatness

$$k_p = \frac{a + b}{2c}; \tag{6.9}$$

- coefficient of grain roundness

$$I = \sqrt[3]{\frac{bc}{a^2}}. \tag{6.10}$$

Their values are derived from the three basic axes, a, b and c. According to the

ratios of axes $b : a$ and $c : b$ and the shape coefficients, the sediment grains can be classified into different shape categories (ZINGG, 1935; NOVÁK, 1985). Figure 81 shows the categories of sediment grains by shape (NOVÁK, 1985). The categories are as follows:

rounded grains: sphere, ellipsoid, disc and rounded rod (Fig. 82);
angular grains: cube, prism, plate and angular rod (Fig. 83).

Rounded bed-load grains have markedly rounded corners and edges. During the course of their long transport down the torrent channel they have been intensively "worked" so that their original shape can no longer be recognised.

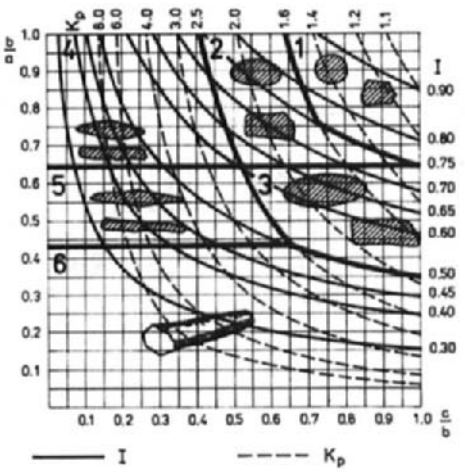


Fig.81 Categorisation of sediment grains by shape (NOVÁK, 1985).

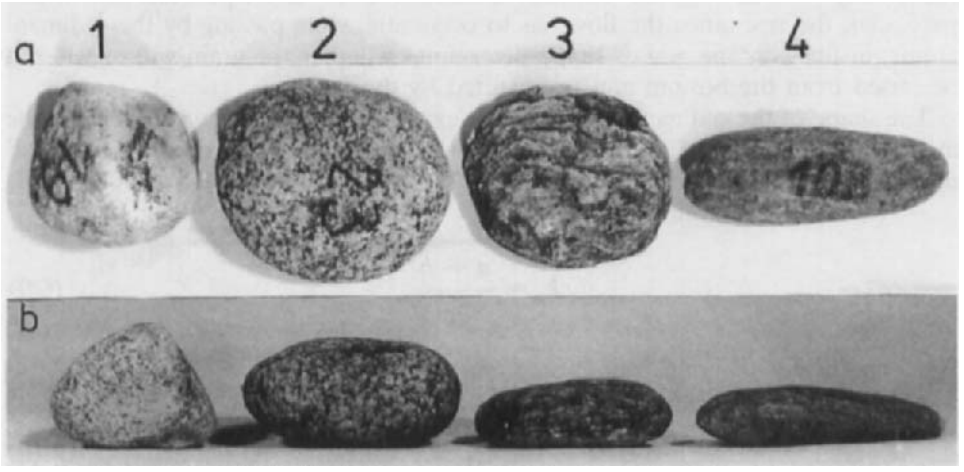


Fig.82 Shape categorisation of rounded sediment grains: a – projection in the plane of axes a and b, b – projection in the plane of axes a and c, 1 – sphere, 2 – ellipsoid, 3 – disk, 4 – rounded rod.

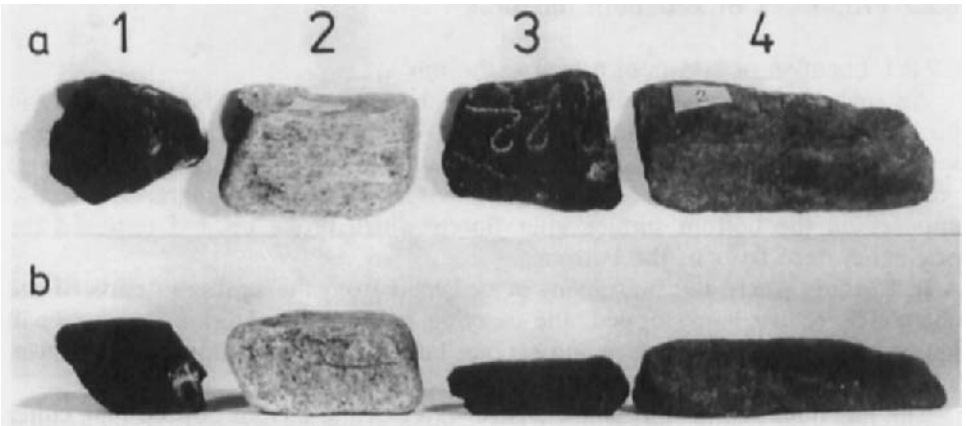


Fig.83 Shape categorisation of angular bottom-load grains: a — projection in the plane of axes a and b, b — projection in the plane of axes a and c, 1 — cube, 2 — prism, 3 — plate, 4 — angular rod.

Sphere-shaped sediment grains have an irregular spherical shape. Their cross-section in the planes of axes *ab*, *ac* and *bc* appears to be an irregular circle to oval.

Ellipsoid is the most common shape of rounded sediment grains in torrents. In cross-section in the planes of axes *ab*, *ac*, *bc*, they have the shape of an irregular ellipse or an irregular ellipse-like figure.

Disc-shaped sediment grains are distinguished from other rounded shapes by their flatness. In cross-section in the planes of axes *ab* they are usually irregularly round, ellipsoidal or oval. In the planes of axes *ac* and *bc* the most frequent cross-sectional shape is an ellipse.

Rounded rod-shaped grains remind one of elongated ellipsoids. Axis *a* is much longer than the other axes in such grains.

Angular bed-load grains include "unworked" grains having sharp edges without rounding, grains with blunt edges and corners and grains with slightly rounded edges with distinct unrounded parts of the original surface.

Cubes are distinguished from the sphere shaped sediment grains by greater angularity and by the formation of flat segments on the surface. The corners and edges are usually blunt.

Prismatic grains have the shape of irregular prisms. Their surface consists of flat "facets", the edges are sharp, blunt or slightly rounded.

Plate-shaped sediment grains differ from other angular grains by a greater flatness. Unlike the discs, their surface consists of more or less flat segments with sharp or blunted corners and edges. The cross-sections in the planes of axes *ab*, *ac* and *bc* usually have a rectangular shape, sometimes they may also be trapezoidal or rhomboidal.

Angular rods are elongated, sometimes irregular, and their corners and edges may be sharp, blunt or slightly rounded, depending on how long they have been transported.

6.2.2 Properties of sediment mixtures

6.2.2.1 Location of sediment grains in the mix

An ungraded bed-load mix develops on the surface of the bottom mainly in torrents with intensive sediment movement. After a period of higher discharges, sediment grains of different size categories (sand, gravel, cobbles) are deposited on gravel bars lying above the level of permanent discharges. The ungraded mix also appears on the bottom surface after floods which broke up and removed the original surface layer of the bottom.

In torrents where the movement of bed load from the upstream parts of the channel is reduced or stopped, the carrying force of flood flows loosens small grains from the bottom surface and exposes larger grains which form a cover layer. The cover layer covers a layer of unsorted sediment mix.

The individual grains forming the cover layer of the torrent bottom may either lie loosely next to one another or may be closely wedged with each other, or may even be anchored in the layer underneath (Fig. 84). A cover layer may also develop between gravel benches where a permanent water flow is concentrated.

In mountain torrents, the sediment mix also contains large boulders which differ markedly in their size from other sediment materials. Either alone or in groups, these boulders stick out through the cover layer or may be entirely buried in the carrying layer of sediment mix (Fig. 85).

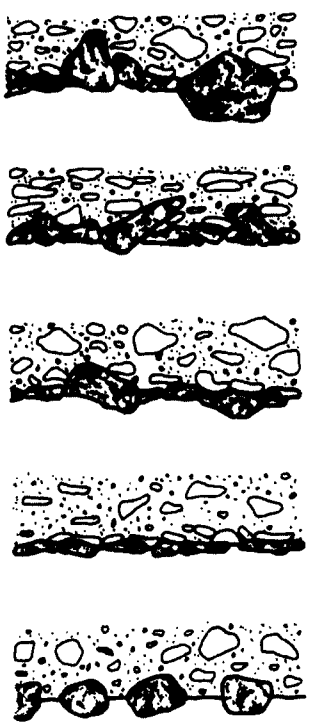


Fig.84 Formation of the cover layer in torrential streams.

Fig.85 Cover layer with large boulders.

The ungraded sediment mix, the cover layer and the cover layer with large boulders form the surface layer of the bottom. The structure of the surface layer of the bottom controls the bottom's roughness and the stability of the sediment. Each particle's resistance or susceptibility to being raised and moved depends on the surface-layer grains' orientation in relation to flow direction. KUKAL (1964) found that the longest axis of the particles carried by the flow is parallel to the direction of the flow: however, when the particle moves by saltation, or is pushed or rolled (at lower flow velocities), its longest axis is perpendicular to the flow direction. A prevalence of particles lying in the surface layer with their longest axis parallel to flow direction is reported by KÜRSTEN (1960), MAREŠ (1987) and NOVÁK (1987).

6.2.2.2 Sediment mix composition

The bed mix forming the surface layer and the layer underneath consists of grains of different sizes. The texture of the mix is best illustrated by the grain-size curve (Fig. 86); the curve shows the grain sizes characterising the structure of the mixture which may be monodispersive, graded, homogeneous, etc.

Monodispersiveness is expressed by the formula:

$$p_1 = \frac{d_{60}}{d_{10}}. \tag{6.11}$$

The bed-load mix is monodispersive if $p_1 < 5$.

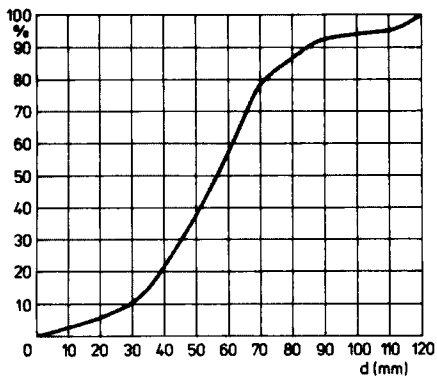


Fig.86 Granularity curve.

The *gradedness of the mixture* is expressed by the formula:

$$p_2 = \sqrt{\frac{d_{75}}{d_{25}}}. \tag{6.12}$$

A well-graded mixture containing the lowest possible number of fractions has a grading coefficient $p_1 \approx 1.5$.

Homogeneity is characterised by the proportions of small and large grains in sediment mix. It is expressed by the homogeneity module:

$$M = \frac{\sum_{o}^{50\%} p_i d_i}{\sum_{50\%}^{100\%} p_i d_i}, \tag{6.13}$$

where p_i – percent proportions of different size fractions,
d_i – grain sizes in the different fractions.

The characteristic grains d_{10}, d_{25}, d_{60}, d_{75} determine what percentage of the total weight of the sample is represented by grains smaller than the given dimension.

6.2.2.2.1 Sediment sampling

The grain-size curve of the sediment is constructed from the analysis of a sample taken from the torrent bottom. The sample should fully represent the structure of the sediment mix in the torret channel segment studied: this must be taken into account when chosing the sampling site. The composition of the sediment mix varies with the gradient of the torrent bottom, with the shape of the channel's cross profile and with the channel's directional characteristics. Hence, the samples should not be taken where large grains (boulders) stick out through the bottom cover layer, e.g., in the concave parts of the bottom, in short bottom segments of a high slope, etc. The best sampling sites are in long straight segments with a uniform bottom slope. A number of samples must be available because the composition of sediment mix also varies with the length of the torrent. ŠKOPEK and NOVÁK (1977) recommend four samples per 1 km.

The bottom surface layer is analysed to assess bottom stability. If there is a cover layer on the bottom surface, the sample should comprise all sediment grains from an area of 1 m^2. The sampling area may be a square, rectangle or circle, or it may have an irregular shape. To get a really representative sample, KREŠL (1980, 1982) recommends a sample strip (at least as wide as the widest grain) across the channel. As to the ungraded surface layer, the sample should be taken from an area of 0.4 × 0.4 m and the sampling probe should be pushed in sufficiently deep to obtain a sample weighing 50 – 100 kg (depending on the size of the largest grain). For surface layers consisting of a cover layer with large boulders, a combination of linear and areal sampling, or areal sampling alone, can be used with advantage. With the combined sampling, the cover layer sample is taken from a 1 m^2 area and the boulders characterising boulder distribution over the whole bottom area are picked along one or two straight lines across the channel. Sectors taken up by

boulders bigger than the largest grain dimension in the cover layer are measured within these lines.

For areal sampling, NOVÁK (1987) recommends picking a representative bottom surface area whose sides should be 5 – 10 m long (Fig. 87) to show the distribution of boulders within the whole bottom. Then a cover-layer sample is taken from a 1 m^2 spot within the representative area. A large slide rule is used to measure the length of axis $b' = d$ of each boulder in the representative area and the boulders' shape is identified.

To determine bed-load movement, a sediment sample is taken from the underlying layer (exposed by removing the cover layer), using a sample probe in the same way as when sampling the ungraded surface layer.

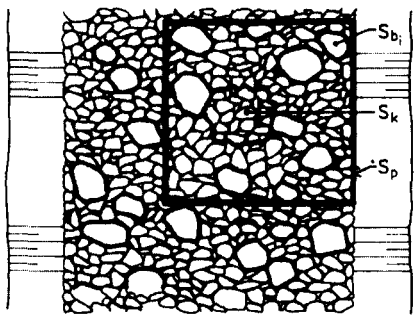


Fig.87 Representative area: surface layers of the bottom, S_p; area of the cover layer, S_k (1 m^2); area of individual boulders, $S_{b,i}$.

6.2.2.2.2 Sediment sample analysis and interpretation

The texture of a sediment sample is determined by including each grain in an appropriate size fraction using sieves or callipers or slide rules. Each fraction is then weighed and its percent proportion of the total sample weight is calculated: these data are needed to construct the grain-size curve.

In the sediment mix, it is important to identify the effective grain which will show the grain size in a homogeneous mix whose behaviour (from the viewpoint of the regularities of sediment mix movement) is the same as that in the actual mix containing grains of different sizes. The effective grain is best characterised by the equation recommended by KRAMER (1935):

$$d_m = \frac{\sum p_i d_i}{100} \quad \text{(m)}. \tag{6.14}$$

This procedure determines the effective grain in the ungraded surface layer, in the cover layer, and in the carrying layer underneath.

For combined sediment sampling from a surface layer consisting of a cover layer and boulders, VESELÝ (1970) recommends identifying the effective grain of the whole surface layer using the formula

$$d_m = \left(\frac{d_{m_k}^2 b_k + d_{m_b}^2 b_b}{b} \right)^{0.5} \quad \text{(m)}, \tag{6.15}$$

where d_{m_k} – effective grain of the cover layer (m),
d_{m_b} – bottom width segment taken up by the cover layer within the chosen sampling line (m),
b_b – bottom width segment taken up by boulders within the chosen sampling line (m),
b – bottom width at the sediment sampling site.

For areal sampling from a surface layer consisting of a cover layer and boulders, NOVÁK (1988) recommends the following procedure:

– identify the cover layer's effective grain d_{m_k} from equation (6.14),
– calculate the mass of each boulder from equation (6.6) or equation (6.7),
– determine the area taken up by the individual boulders S_b using the following formula:

$$S_b = k_2 d^2 \quad \text{(m}^2\text{)}, \tag{6.16}$$

where k_2 – shape coefficient for the corresponding boulder shape (from Tab. 57),
d – measured dimension of the boulder,

– calculate the total area the cover layer takes up in the surface layer within the representative area S_p from the following equation:

$$S_k = S_p - \sum S_b \quad \text{(m}^2\text{)}, \tag{6.17}$$

– convert the mass of the individual size fractions of the cover layer, sampled from a 1 m^2 area, to the mass figure comprising the whole cover layer S_k,
– determine the proportion of the mass of every size fraction that occurs in the representative area (i.e., the mass of all boulders and the cover layer in the whole area S_k) as a percentage of the mass of the whole sediment sample,
– use equation (6.14) to identify the effective grain d_{m_p} of the whole area.

Example 1.

a) Select a representative rectangular area 4 × 7 m in size; $S_p = 28$ m^2.

b) The area in the surface layer of the bottom taken up by large boulders (Tab. 54) is $\sum S_b = 6.66$ m^2.

c) The surface layer area taken up by the cover layer is $S_k = S_p - \sum S_b = 28.0 - 6.66 = 21.34$ m.

d) Analysis of the texture of the cover layer of a 1 m^2 area is shown in Tab. 55 and the grain-size curve is shown in Fig. 86. The effective grain of the cover layer is $d_{m_k} = \frac{19.66}{100} = 0.22$ m.

e) Analysis of the texture of the boulders is shown in Tab. 54 and the grain-size curve is in Fig. 88. The effective grain of the boulders is $d_{m_b} = \frac{47.38}{100} = 0.47$ m.

f) Analysis of the texture of the whole surface layer, including the 1 m² sample of the cover layer (converted to the whole area S_k) and all boulders, is shown in Tab. 56 and the grain-size curve is in Fig. 88.

g) The effective grain of the whole surface layer is $d_{mp} = \frac{35.71}{100} = 0.36$ m.

h) The percentiles derived from the grain-size curve for the whole surface layer are: $d_{10} = 0.11$ m, $d_{50} = 0.34$, $d_{60} = 0.38$, $d_{90} = 0.6$ m.

Table 54

Analysis of the grain sizes of separate large boulders

Grain diameter	Number of grains	Weight	Percentual proportions		Ordinate of the granularity curve	Grain shape	Shape coefficient	Grain surface area
d m	pcs	G kg	p %	pd			k_2	S_b m²
0.7	1	167.83	4.67	3.27	100	sphere	0.900	0.44
0.67	1	209.90	5.84	3.91	95.33	ellipse	1.174	0.53
0.65	1	272.73	7.59	4.93	89.49	sphere	0.900	0.38
0.60	2	311.69	8.67	5.20	81.90	sphere	0.900	0.64
0.55	1	169.23	4.71	2.59	73.23	sphere	0.900	0.27
0.54	1	146.01	4.06	2.19	68.52	sphere	0.900	0.26
0.50	1	232.17	6.46	3.23	64.46	ellipse	1.174	0.29
0.48	2	214.44	5.96	2.86	58.00	sphere	0.900	0.42
0.45	3	333.03	9.26	4.17	52.04	sphere	0.900	0.54
0.42	2	193.85	5.39	2.26	42.78	ellipse	1.174	0.41
0.40	3	319.68	8.89	3.56	37.39	sphere	0.900	0.42
0.38	1	56.69	1.58	0.60	28.50	sphere	0.900	0.13
0.36	1	69.05	1.92	0.69	26.92	ellipse	1.174	0.15
0.35	4	244.85	6.81	2.38	25.00	ellipse	1.174	0.57
0.34	2	161.68	4.49	1.53	18.19	sphere	0.900	0.20
0.33	2	108.31	3.01	0.99	13.70	sphere	0.900	0.20
0.32	1	29.84	0.83	0.27	10.69	ellipse	1.174	0.12
0.30	3	138.26	3.85	1.16	9.86	sphere	0.900	0.24
0.28	2	51.24	1.43	0.40	6.01	sphere	0.900	0.14
0.27	3	116.27	3.23	0.87	4.58	sphere	0.900	0.21
0.24	1	24.26	0.67	0.16	1.35	sphere	0.900	0.05
0.23	1	24.51	0.68	0.16	0.68	sphere	0.900	0.05
Σ		3595.52	100	47.38				6.66

Table 55

Analysis of the grain sizes in the cover layer in an area of 1 m²

Grain diameter	Weight	Percentual proportion		Ordinate of the grain-size curve
d	*G*	*p*	*dp*	
m	kg	%		%
0.34	25.36	20.63	7.01	100.00
0.27	12.50	10.17	2.75	79.37
0.24	10.10	8.22	1.97	69.20
0.22	12.40	10.09	2.22	60.98
0.15	6.30	5.12	0.77	50.89
0.14	5.80	4.71	0.66	45.77
0.13	14.00	11.39	1.48	41.06
0.12	10.15	8.26	0.99	29.67
0.10	8.40	6.83	0.68	21.41
0.09	5.30	4.31	0.39	14.58
0.08	6.25	5.08	0.41	10.27
0.07	3.15	2.56	0.18	5.19
0.06	1.80	1.46	0.09	2.63
0.05	1.20	0.98	0.05	1.17
0.04	0.23	0.19	0.01	0.19
$\sum$	122.94	100.00	19.66	

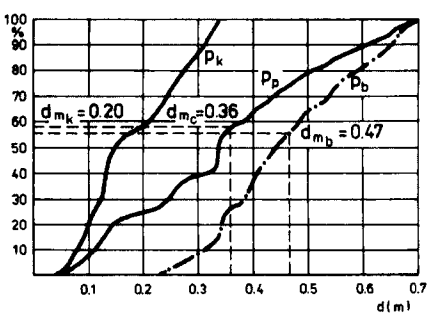


Fig.88 Granularity curve of the sediment in the upper layer of the bottom: p_k — cover layer, P_b — boulders, p_p — whole surface layer.

Table 56

Analysis of the grain sizes in the whole surface layer

Grain diameter	Weight	Percentual proportion		Ordinate of the grain-size curve
d	*G*	*p*	*dp*	
m	kg	%		%
0.70	167.83	2.70	1.89	100.00
0.67	209.90	3.37	2.26	97.30
0.65	272.73	4.39	2.85	93.93
0.60	311.69	5.02	3.01	89.54
0.55	169.23	2.72	1.50	84.52
0.54	146.01	2.35	1.27	81.80
0.50	232.17	3.73	1.87	79.45
0.48	214.44	3.45	1.66	75.72
0.45	333.03	5.35	2.41	72.27
0.42	193.85	3.12	1.31	66.92
0.40	319.68	5.14	2.06	63.80
0.38	56.69	0.91	0.35	58.66
0.36	69.05	1.12	0.40	57.75
0.35	244.85	3.94	1.38	56.63
0.34	702.86	11.30	3.84	52.69
0.33	108.31	1.74	0.57	41.39
0.32	29.84	0.48	0.15	39.65
0.30	138.26	2.22	0.67	39.17
0.28	51.24	0.82	0.23	36.95
0.27	383.02	6.16	1.66	36.13
0.24	239.79	3.86	0.93	29.97
0.23	24.51	0.39	0.09	26.11
0.22	264.62	4.25	0.94	25.72
0.15	134.44	2.16	0.32	21.47
0.14	123.77	1.99	0.28	19.31
0.13	298.76	4.80	0.62	17.32
0.12	216.60	3.48	0.42	12.52
0.10	179.26	2.88	0.29	9.04
0.09	113.10	1.82	0.16	6.16
0.08	133.38	2.14	0.17	4.34
0.07	67.22	1.08	0.08	2.20
0.06	38.41	0.62	0.04	1.12
0.05	25.61	0.42	0.02	0.50
0.04	4.91	0.08	0.01	0.08
$\sum$	6219.06	100.00	35.71	

6.2.3 Incipient sediment movement

Sediments lying in the surface layer of the bottom are exposed to the force of the water flow: this force includes hydrodynamic pressure and lift forces. Each particle's resistance to being raised and moved depends on its size and shape and on its position in the sediment mix. The particle begins to move when the active forces exceed the value of this resistance.

KRAMER (1935) distinguishes three types of movement:

- *slight movement*: a small amount of bed-load material moving on the bottom,
- *medium movement*: medium size grains starting to move,
- *sweeping movement*: whole mix in motion.

When defining a stable bottom, there should be no sweeping movement of the mix. As a heterogeneous sediment mix can be replaced by a monodispersive mix in which the grain dimension is d_m, it can be laid down as a condition that the sediment movement starts as movement of individual particles corresponding to the size of the effective grain.

Mass movement of bed-load material occurs following the incipient movement of the individual sediment grains forming the surface layer of the bottom, which are most exposed to the action of the water current under the given flow conditions and which at the same time have the least stable position (Fig. 89). While these

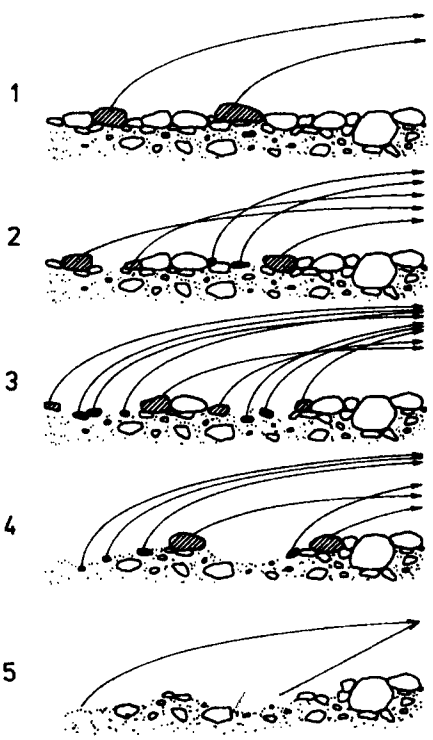


Fig.89 Schematic drawing of the successive removal of grains from the surface layer of the bottom.

grains are washed away, others, which were originally stable in the surface layer of the sediment material, are denuded and become exposed to direct action of external forces. During the first stage, the material that is set in motion will involve not only the small particles but also the larger ones which are unstable. The bonds between the grains in the cover layer must have been broken before the small grains, wedged between the larger sediment particles, can abruptly be seized and carried off by the current. The surface layer of the bottom, consisting of coarser material, is progressively dismantled while much smaller particles are removed from the layer underneath for which the strength of the current abruptly becomes more than critical. The movement at which the surface cover layer loses its resistance is the onset of mass movement of the sediment material and the torrent's bottom deepens.

The theory of the start of movement of bed-load material relies mainly on methods based on critical bed velocities and critical shear stress.

6.2.3.1 The critical bed velocity method

According to NOVÁK (1985), the balance between the active forces F_x and passive forces F_o (Fig. 90) provides an adequate expression of the sediment particles' resistance to being raised and moved. A sediment grain is stable if

$$F_x = F_o . \tag{6.18}$$

The forces that act on the sediment particles include:
hydrodynamic pressure force:

$$F_d = \varrho C_x k_1 d^2 \frac{u_d^2}{2} = F_x \qquad \text{(N)}; \tag{6.19}$$

hydrodynamic lift force:

$$F_z = \varrho C_y k_2^2 d^2 \frac{u_d^2}{2} \qquad \text{(N)}; \tag{6.20}$$

gravity of the submerged bed load grain:

$$F_G = k_3 d^3 (\varrho_m - \varrho)\, g \qquad \text{(N)}. \tag{6.21}$$

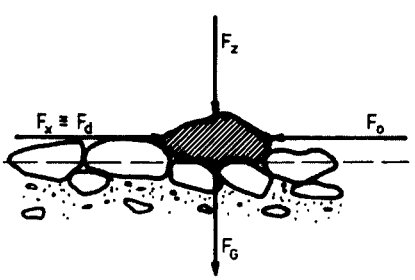


Fig.90 Forces acting on an individual sediment grain.

In these equations:

C_x – drag coefficient,
C_y – lift coefficient,
k_1 – sediment particle shape coefficient – horizontal,
k_2 – sediment particle shape coefficient – vertical,
k_3 – sediment particle volume coefficient,
d – sediment particle dimension (m),
u_d – bed velocity ($m \cdot s^{-1}$),
ϱ_m – bed-load material density ($kg \cdot m^{-3}$),
ϱ – water density ($kg \cdot m^{-3}$),
g – gravity due to acceleration ($m \cdot s^{-2}$).

The force of resistance acting against the movement of the sediment particles is:

$$F_o = (F_G - F_Z) f = \left[k_3 d^3 (\varrho_m - \varrho) g - \varrho C_y k_2 d^2 \frac{u_d^2}{2} \right] f, \tag{6.22}$$

where f – coefficient of resistance of the bottom to the sediment particles' being raised and moved.

Introducing equations (6.19) and (6.22) into equation (6.18) will make:

$$\varrho C_x k_1 d^2 \frac{u_d^2}{2} = \left[k_3 d^3 (\varrho_m - \varrho) g - \varrho C_y k_2 d^2 \frac{u_d^2}{2} \right] f. \tag{6.23}$$

This equation can be rearranged to obtain the formula for the critical bed velocity:

$$u_{d_v} = \left(\frac{k_3 \, \mathrm{d} (\varrho_m - \varrho) f \cdot 2 g}{\varrho (k_1 C_x + k_2 C_y f)} \right)^{0.5} \tag{6.24}$$

By factoring the constant values out to the common constant, equation (6.24) can be rearranged to obtain

$$u_d = C d^{0.5}, \tag{6.25}$$

where

$$C = \left(\frac{k_3 (\varrho_m - \varrho) f \cdot 2g}{\varrho (k_1 C_x + k_2 C_y f)} \right)^{0.5} \tag{6.26}$$

NOVÁK (1987) analysed the sediment mixes from the torrents of the Czech Republic to determine the shape coefficients k_1, k_2 and k_3. Coefficients k_1 and k_2 serve to convert the area exposed to the hydrodynamic pressure or hydrodynamic lift (calculated as the area of a square whose side is equal to the dimension of the sediment grain) to a value corresponding to the cross-sectional area within plane *ac* or *ab* in the actual sediment particle.

The values of these coefficients are calculated from the following two equations:

$$k_1 = \frac{S_h}{d^2} \tag{6.27}$$

and

$$k_2 = \frac{S_Z}{d^2}, \tag{6.28}$$

where S_h = area exposed to force F_d,
S_Z = area exposed to force F_Z.

Coefficient k_3 expresses the relationship between the volume of the actual sediment grain and a cube whose side is equal to the grain dimension d:

$$k_3 = \frac{V}{d^3}. \tag{6.29}$$

Values of coefficients k_1, k_2 and k_3 for the different shape categories are shown in Tab. 57 and for different rocks in Tab. 58.

Coefficient f showing the effect of friction between sediment particles and the effect of adjacent sediment grains on the bottom surface layer's resistance to the sediment particle's being raised and moved is computed from the equation

$$f = \frac{F}{F_G - F_Z}, \tag{6.30}$$

where F – force needed to raise and move the sediment grains.

The data on coefficient f for different kinds of rock are shown in Tab. 58.

The drag coefficient C_x depends on sediment grain size and on the value of the Reynolds number. The resistance coefficient values for a Reynolds number common for turbulent flows in torrent channels ($R_e = 1.77 \times 10^4 + 3.83 \times 10^4$) were experimentally determined by measurement in a hydraulic channel (NOVÁK, 1987) and are shown in Tab. 57 for the different shape categories and in Tab. 58 for different rocks.

Table 57

Coefficients k_1, k_2, k_3 and C_x for different shape categories

Rounded grains				
Category	k_1	k_2	k_3	C_x
Sphere	0.735	0.900	0.367	0.49
Ellipsoid	0.761	1.174	0.417	0.45
Disk	0.502	1.207	0.292	0.39
Angular grains				
Category	k_1	k_2	k_3	C_x
Cube	0.721	0.898	0.350	0.62
Prism	0.776	1.123	0.399	0.64
Plate	0.517	1.220	0.298	1.09

Table 58

Values of coefficients k_1, k_2, k_3, f, C_x and values of constant C for different rocks

Rock	k_1	k_2	k_3	ϱ_m	f	C_x	C
Granite	0.726	1.017	0.418	2620	1.36	0.46	5.58
Silica porphyry	0.671	1.016	0.363	2580	1.38	0.56	4.98
Migmatite	0.675	1.134	0.368	2690	1.35	0.44	5.38
Gneiss	0.738	1.290	0.384	2630	1.27	0.48	3.91
Granodiorite	0.687	1.153	0.358	2700	1.46	0.56	5.05
Granulite	0.684	1.120	0.353	2630	1.39	0.58	4.83
Basalt	0.758	1.131	0.375	2860	1.48	0.45	5.63
Slate	0.696	1.256	0.340	2680	1.40	0.83	4.20
Mica schist	0.591	1.161	0.349	2710	1.24	0.46	5.24
Phyllite	0.592	1.225	0.340	2700	1.34	0.46	5.19
Sandstones, permocarb.	0.666	1.031	0.371	2550	1.14	0.47	4.96
Sandstones, flysh	0.659	1.194	0.361	2550	1.20	0.49	4.77
Sandstones, chalk	0.640	1.125	0.359	2000	1.54	0.50	4.15
Greywacke	0.705	1.391	0.335	2680	1.31	0.51	4.60
Marlites	0.532	1.025	0.290	2470	1.39	0.58	4.55

The coefficient of hydrodynamic lift C_y for the sediment grains was determined to be $C_y = 0.178$ (EINSTEIN and EL SAMNI, 1949) and it applies to all categories of sediment material.

Determination of the different parameters of equation (6.26) allowed of establishing constants C for different types of rock (NOVÁK, 1987). The data are shown in Tab. 58.

Data from Table 58 can be used in areas for which the constant C has not yet been determined; however, this can only be so if grains coming from a given rock prevail in the sediment material. If the material contains larger proportions of grains coming from different rocks, the sample grains must be categorised by shape and their average value for the mixture under study should be identified, using the different coefficient values from Tab. 57; thus, for example, for coefficient C_x

$$C_x = \frac{\sum C_{x_i} n_i}{n}, \tag{6.31}$$

where C_{x_i} – drag coefficient for the respective shape categories,
n_i – number of grains of the respective shape category,
n – total number of grains in the sediment sample analysed.

The same procedure is used to determine the other coefficients; rock density can also be specified, using the data from Tab. 51. The coefficient's average value $f = 1.27$ is used in such a case for the determination of the constant C. With a lower accuracy, the following average values of this constant can be used:

– $C = 5.38$ for bed-load mixes in which rounded grains prevail, the proportion of angular grains being lower than 15%,

– $C = 4.59$ for bed-load mixes in which the proportion of angular grains is larger than 15%.

The critical bed velocities have also been studied by other authors; however, the only critical bed velocity equation applicable to very rough torrent channels is that developed by THIÉRY (1891):

$$u_{d_v} = \left(\frac{d(\varrho_m - \varrho) f \cdot 2g}{0.076\varrho}\right)^{0.5} \qquad (\mathrm{m \cdot s^{-1}}), \tag{6.32}$$

where f – coefficient of friction.

6.2.3.2 The critical average channel velocity method

A number of equations have been developed for the determination of the critical average channel velocity but only two (SRIBNYI, 1932 and NEILL, 1967) are applicable to the defining of the onset of movement of a torrent's sediments.

The SRIBNYI equation is as follows:

$$v_v = 5d_m^{0.5} \qquad (\mathrm{m \cdot s^{-1}}). \tag{6.33}$$

The NEILL equation is as follows:

$$v_v = \left[2.5\left(\frac{h}{d_m}\right)^{0.2} g\left(\frac{\varrho_m - \varrho}{\varrho}\right) d_m\right]^{0.5} \qquad (\mathrm{m \cdot s^{-1}}). \tag{6.34}$$

Critical average channel velocities were also determined experimentally. Fig. 91 shows a nomogram for determining the critical velocities by the former Soviet Standard St-24-2396.

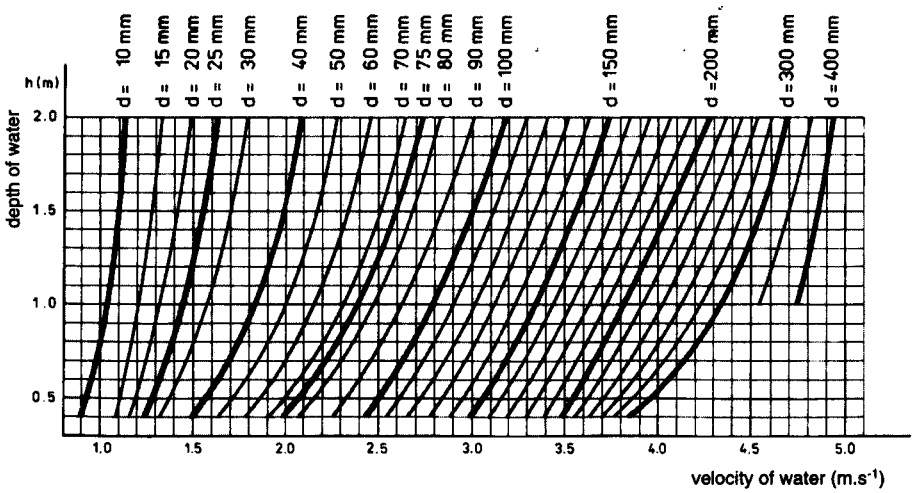


Fig.91 Critical average channel velocities according to St-24-2396.

6.2.3.3. The critical shear stress method

The incipient sediment movement can also be defined using the critical shear stress. The sediment grains remain stable as long as the shear stress, arising during the design discharge

$$\tau_o = \varrho Rig \qquad \text{(Pa)}, \tag{6.35}$$

is smaller than or equal to the critical shear stress.

A number of equations have been derived for the determination of critical shear stress. We have chosen only those applicable to torrents.

SHIELDS (1936) conducted laboratory experiments to define the incipient sediment movement; he derived the following equation for critical shear stress:

$$\tau_c = f\left(\frac{d\sqrt{gRi}}{\nu}\right)(\varrho_m - \varrho)\, dg \qquad \text{(Pa)}, \tag{6.36}$$

where ν – coefficient of kinematic viscosity of water.

While developing his equation, Shields found that the quotient $\frac{\tau_c}{(\varrho_m - \varrho)\, dg}$ is not constant, the function $f\frac{d\sqrt{gRi}}{\nu}$ ranging from 0.03 to 0.06.

It can be seen in Fig. 92 that for torrent streams of a higher R_e this function's value is 0.06. Then equation (6.36) will be

$$\tau_c = 0.06(\varrho_m - \varrho)\, d_m g \qquad \text{(Pa)} \tag{6.37}$$

As to other equations for critical shear stress, EGIAZAROV's (1965) equation proved applicable to torrential flows:

$$\tau_c = \frac{0.1(\varrho_m - \varrho)\, d_{50} g}{\left(\log 19 \cdot \frac{d_{50}}{d_m}\right)^2}. \qquad \text{(Pa)} \tag{6.38}$$

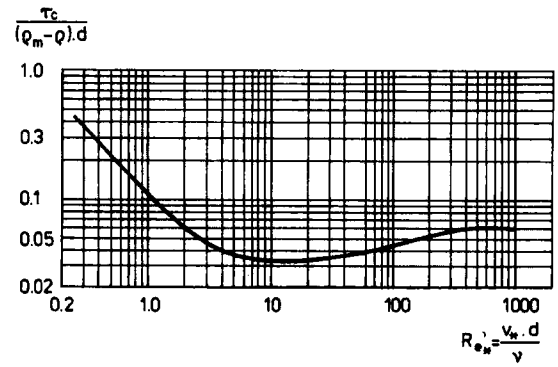


Fig.92 Shields' diagram: dependence of the critical shear stress on the shear Reynolds number (after GRAF, 1971).

6.2.4 Movement of sediment material

The critical values of water flow along the torrent bed, i.e., the critical bed velocity or critical average channel velocity, or the critical shear stress, having been exceeded, the individual sediment grains start to move. The sediment particles begin shifting and rolling without leaving the surface of the bottom. During this rolling movement, the velocity of the sediment particles depends on the morphology (unevenness) of the bottom surface. The size of projections on the bottom influences the speed of the small particles more than that of the larger ones: the small particles move much more slowly. Irregular coarseness of the bottom surface slows down or stops the particles' movement. Any obstacle in a particle's way can slow down or speed up its forward movement and, at the same time, increase the velocity of its upward movement $\left(\frac{F_Z}{F_G} > 1\right)$, which means that the rolling motion changes into saltation. The path of this type of movement is shown in Fig. 93.

As soon as movement in the direction $(F + G)$ begins, the increase in the sediment particles' movement reduces the value of the F parameter. Consequently, the resultant $(F + G)$ declines towards the bottom. When the path of a particle reaches the top position y_{max}, its speed further increases while vector F declines continuously and turns. The curvature of the particle's path takes a decreasing value of the landing angle β. After landing on the bottom the whole process takes place again: hence, the particle moves on by jumping (saltation).

Particles moving on by shifting, rolling or saltation are referred to as bed-load particles or shifting particles. Very small grains are first quickly raised above the bottom and then carried forward by the turbulent flow: such particles are referred to as floating particles.

6.2.4.1 Bed load discharge

The amount of material that flows along the torrent channel can be calculated from equations based on both the bed-load movement theory and experimental results. Methods developed by KALINSKE (1947), MEYER-PETER (1949), EINSTEIN (1942, 1950) and JÄGGI (1983) are suitable for the purposes of torrent control, e.g., for defining the required space in settling reservoirs, etc.

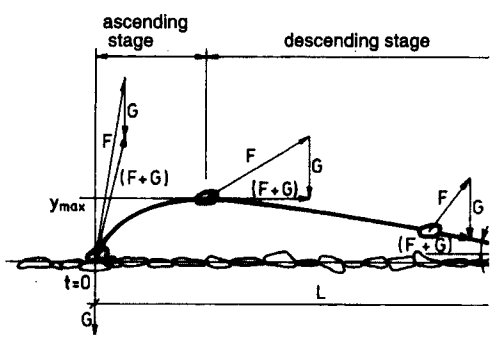


Fig.93 Saltation movement of a bed-load grain.

KALINSKE (1947) expressed the unit bed load discharge by the following equation:

$$q_{jG} = \frac{2}{3} \cdot 0.35 u_* d_m \frac{\overline{(u_d - u_{d_v})}}{u_*} \qquad (\mathrm{m^3 \cdot s^{-1} \cdot m^{-1}}), \tag{6.39}$$

where u_* – shear velocity: see equation (6.66),
u_d – bed velocity $(\mathrm{m \cdot s^{-1}})$,
u_{d_v} – critical bed velocity $(\mathrm{m \cdot s^{-1}})$.

Equation (6.39) can be rearranged:

$$\frac{q_{j_s}}{u_* d_m} = f\left(\frac{\tau_c}{\tau_o}\right) \tag{6.40}$$

This relationship is illustrated in Fig. 94.

MEYER-PETER conducted a two-year laboratory investigation which he summarised in an equation based on the condition of maintaining a state of equilibrium in the bed-load process, i.e., on the equality of the amount of sediment material carried away from the segment investigated and the amount of such material brought to that segment from outside:

$$\varrho R_d \left(\frac{k_d}{k_s}\right)^{3/2} i = 0.047(\varrho_m - \varrho)\, d_m + 0.25 \left(\frac{\varrho}{g}\right)^{1/3} q_{jG}^{2/3}, \tag{6.41}$$

where R_d – hydraulic radius corresponding to the part of flow influencing the movement of sediment,
k_d – Strickler's velocity coefficient due to the bottom,
k_s – Strickler's velocity coefficient due to the sediment mix,
i – energy line slope (bed slope).

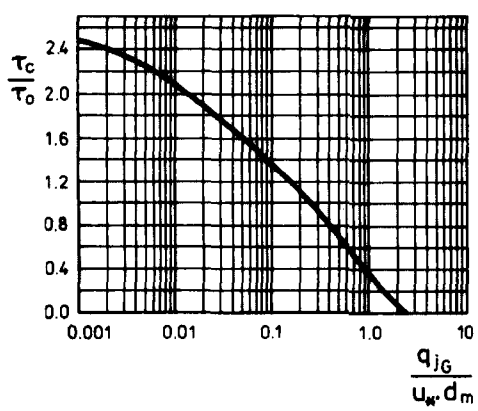


Fig.94 Graphical representation of the equation for specific bed-load discharge (after KALINSKE, 1947).

The left side of this equation shows the shear stress arising on the surface of the bottom and constituted only by the flow passing above the bottom. Hence, the factors taken into account in the shear-stress equation include the hydraulic radius R_d and only the part of energy losses that is left with the individual grains of the sediment mix after subtraction of the losses caused by the irregularities and roughness of the bottom surface.

The hydraulic radius R_d, corresponding to the part of the flow influencing the movement of the sediment material, can be expressed by the formula

$$R_d = \frac{S_d}{O_d} \quad \text{(m)}, \tag{6.42}$$

where S_d – area of the flow profile (waterway) corresponding to flow above the bottom: this area is delineated by the bottom, the lines bisecting the angle formed by the bottom and the channel slope, and by the surface of the design flow (Fig. 95) (m^2),

O_d – wetted perimeter of the bottom (m).

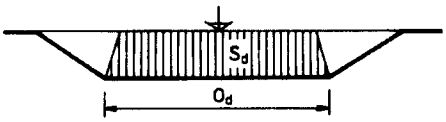


Fig.95 Delineation of the flow area S_d corresponding to flow above the bottom.

The slope of the energy line for the bottom as a whole can be expressed using the Strickler velocity formula; the equation will be

$$i = \frac{v^2}{k_d^2 R_d^{4/3}} \tag{6.43}$$

and the energy line slope for the sediment mix is expressed by the equation

$$i_s = \frac{v^2}{k_s^2 R_d^{4/3}}. \tag{6.44}$$

Comparison of equations (6.43) and (6.44), the exponent being adjusted according to experimental results, will yield the value of the part of the whole energy line gradient which is due to the sediment mix:

$$i_s = i \left(\frac{k_d}{k_s}\right)^{3/2}. \tag{6.45}$$

It is difficult to determine the $k_d : k_s$ ratio in this equation. ZELLER (1963) believes that $k_d : k_s$ ranges from 0.5 for irregularly formed bottoms to 1.0 for flat bottoms.

Equation (6.41) can be rearranged for the specific bed-load discharge. Using equation (6.42), the following relationship can be obtained:

$$q_{jG} = \left\{0.86 \cdot \left[\varrho \left(\frac{k_d}{k_s}\right)^{3/2} R_d i - 0.047(\varrho_m - \varrho)\, d_m\right]\right\}^{3/2} \quad (\mathrm{kg \cdot s^{-1} \cdot m^{-1}}). \quad (6.46)$$

This equation was developed under the following assumptions: $i = 0.4 - 20‰$, $d_m = 0.0004 - 0.03$ m, $h = 0.01 - 1.2$ m, $Q = 0.002 - 2.0\ \mathrm{m^3 \cdot s^{-1}}$.

JÄGGI (1984) found that the mechanism of movement of bed-load material in the steep channels of torrents at slopes of 30–200‰ does not differ from movement observed at lower gradients. He added new data to Meyer-Peter's results and derived an equation for the specific discharge of bed load:

$$q_{jG} = 2.5 q i^{0.6} \left(i - \frac{d_m}{12.1 h_m}\right) \quad (\mathrm{m^3 \cdot s^{-1} \cdot m^{-1}}), \quad (6.47)$$

where q – specific discharge $(\mathrm{m^3 \cdot s^{-1} \cdot m^{-1}})$,
i – energy line slope,
h_m – mean depth (m).

EINSTEIN (1942, 1950) based his equation of the movement of bed-load material on the finding that the movement of the individual particles is not continuous: it is interrupted by a rest period during which the particle remains deposited in the bed-load layer. The state of equilibrium is attained when the amount of particles raised and removed (eroded) from the bottom is the same as the amount of particles deposited on the bottom.

The amount of particles eroded from a unit area of the bottom per unit time is

$$n_e = \frac{\Delta P_d p}{k_2 d^2 k_4} \sqrt{\frac{g(\varrho_m - \varrho)}{d\varrho}}. \quad (6.48)$$

The amount of particles settled in a unit area of bottom per unit time is

$$n_u = \frac{\Delta p_s q_{iG}}{A_L k_3 d^4 \varrho_m g}. \quad (6.49)$$

In these equations:
ΔP_d – the given grain fraction's proportion of the surface layer of the bottom,
ΔP_s – the same fraction's proportion of grains in motion,
p – probability that the particle will move,
k_2 – coefficient of grain shape,
k_3 – coefficient or grain volume,
k_4 – constant of the timescale,
A_L – constant of the unit path of grain movement.

The basic equation for bed-load material movement is derived from the equality of equations (6.48) and (6.49):

$$\frac{\Delta P_s q_{jG}}{A_L k_3 d^4 \varrho_m g} = \frac{\Delta P_d p}{k_2 k_4 d^2} = \sqrt{\frac{g(\varrho_m - \varrho)}{d\varrho}}. \quad (6.50)$$

Introduction of the average distance of movement of the individual particle

$$A_L d = \frac{\lambda d}{1 - p} \tag{6.51}$$

in equation (6.50) will produce an equation in the following form:

$$\frac{p}{1 - p} = \left(\frac{k_2 k_4}{k_3 \lambda}\right)\left(\frac{\Delta P_s}{\Delta P_d}\right)\left(\frac{q_{jc}}{\varrho_m g}\sqrt{\frac{\varrho}{\varrho_m - \varrho}}\sqrt{\frac{1}{gd^3}}\right), \tag{6.52}$$

where λ – constant whose approximate value is 100.

The part of equation (6.52) to which Einstein refers as the intensity of sediment transport is

$$\Phi = \frac{q_{jc}}{\varrho_m g}\sqrt{\frac{\varrho}{(\varrho_m - \varrho)\, gd^3}}. \tag{6.53}$$

The probability that a sediment particle will be raised from the bottom and set in motion means, at the same time, the probability that the particle's dynamic buoyancy will be greater than its weight under water

$$\varrho C_y k_2 \cdot \frac{1}{2} u_d^2 d^2 (1 + \eta) > k_3 d^3 (\varrho_m - \varrho)\, g\,, \tag{6.54}$$

where η – parameter expressing the change of velocity with time.

The bed velocity u_d can be expressed by the equation

$$u_d = (5.75 \cdot \log 10.6 X/\Delta)\sqrt{R_d i g}\,, \tag{6.55}$$

where X – 0.77,

Δ – height of the projections forming the coarseness of the bottom.

Introduction of equation (6.55) in equation (6.54) will allow us to express the probability of the particle's movement:

$$l > \left(\frac{1}{1 + v}\right)\left(\frac{\varrho_m - \varrho}{\varrho}\,\frac{d_{35}}{R_d i}\right)\left(\frac{2k_3}{0.178 k_2 \cdot 5.75^2}\right)\left(\frac{1}{\log^2 10.6X/\Delta}\right). \tag{6.56}$$

The part of this equation to which Einstein refers as flow intensity is:

$$\Psi = \frac{\varrho_m - \varrho}{\varrho}\,\frac{d_{35}}{R_d i}. \tag{6.57}$$

The specific discharge of bed-load material can be determined from equation (6.53):

$$q_{jG} = \frac{\Phi \varrho_m g}{\sqrt{\frac{\varrho}{(\varrho_m - \varrho)\, g d_{35}^3}}} \qquad (\mathrm{kg \cdot s^{-1} \cdot m^{-1}}). \tag{6.58}$$

The Φ value is determined from the relationship between Φ and Ψ, as shown in Fig. 96.

The equations for bed-load discharge express the maximum capacity of the flow to transport the eroded material, i.e., the "transport capacity" of the channel. The amount of material eroded from the watershed and from the water course's channel can be either larger or smaller than the transport capacity of the channel and can be determined using the equations for sediment runoff.

6.2.4.2 Sediment runoff

ŠKOPEK (1987) recommends using *Gavrilovič's* (1972) *formula* to determine the annual average amount of transported sediments material. The Gavrilovič formula is:

$$Q_G = THP(Z)^{3/2} SR_n \qquad (\mathrm{m^3 \cdot year^{-1}}), \tag{6.59}$$

where T – temperature characteristics,
H – average value of total annual precipitation (mm),
P – 3.14,
Z – factor of the watershed's susceptibility to erosion,
S – watershed area ($\mathrm{km^2}$),
R_n – coefficient of retention and retardation.

The temperature characteristics are determined from the equation

$$T = (0.1 t_o + 0.1)^{1/2}, \tag{6.60}$$

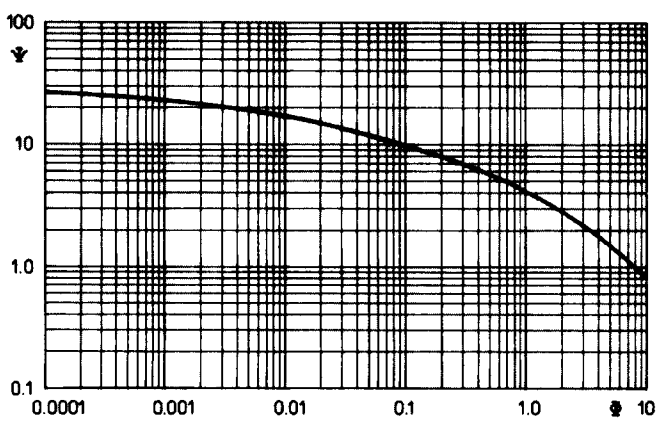


Fig.96 Dependence of ψ on φ (after EINSTEIN, 1950).

where t_o – average annual temperature.

For the factor of watershed's susceptibility to erosion LAZAREVICH (1983) established the following formula:

$$Z = XP(E + I^{1/2}), \tag{6.61}$$

where X – coefficient expressing the effect of the watershed's vegetation cover type and of the mode of watershed management on the erodibility of the soils (tab. 59),
P – coefficient of the mean permeability of the soils (see Tab. 48),
E – coefficient of the watershed's susceptibility to erosion (see Tab. 49),
I – average gradient of slopes in the watershed (%).

The coefficient of retention and retardation R_n expresses the effect of the surface geomorphology of the watershed on the amount of sediment deposited in torrent segments of a lower bottom gradient or in widened valleys as a result of a sudden drop in the water current's carrying capacity. Hence, in the profile measured, this coefficient can be calculated from the equation

$$R_n = \frac{(OV_s)^{1/2}}{0.25(L + 10)}. \tag{6.62}$$

The value of the average annual amount of transported material is used in assessing the silting of settling check dams or in the stream channels.

The quantity of sediments raised and transported from the watersheds during floods (Q_{100}) is well determined by the equation suggested by CHERCHEULIDZE (1947):

$$Q_G = \frac{2m\varphi Q}{\gamma_m} \quad (\mathrm{m^3 \cdot s^{-1}}), \tag{6.63}$$

where m – coefficient depending on the value of the watershed's erosion susceptibility factor Z (Tab. 60),
φ – coefficient depending on stream bottom slope (Tab. 61),
Q – discharge ($\mathrm{m^3 \cdot s^{-1}}$),
γ_m – specific weight of the sediment material.

Table 59

Coefficient X showing the effect of the type of vegetation cover in the watershed on erosion intensity

X	Characteristics of soil surface
1.0	exposed soil surface
0.9–0.8	arable land, unsown
0.6	root crops
0.5	degraded pastures
0.4	thin tree and shrub stands
0.3–0.2	cultivated meadows and other cultivated grassland
0.1	forests of a suitable species composition

For the determination of the total amount of sediment material transported from the watershed during a flood, ŠKOPEK (1987) recommends basing the calculation on the time (duration) of the precipitation causing maximum flow: in mountain streams this duration is about 2/3 of that of the critical precipitation. The portion of particles suspended in water (not greater than 20% of the total volume) should be subtracted from the total amount of bed-load material calculated (JAŘABÁČ and CHLEBEK, 1984).

Example 2.

a) Bed-load material discharge at Q_{100} under the following hydraulic conditions in the upstream part of the torrent above the erosion-control structure: $Q_{100} = 22.8\ m^3 \cdot s^{-1}$, flood wave flow duration $t = 2916$ s, $b = 5.0$ m, $h = 2.0$ m, $i = 0.087$, $d_m = 0.12$ m, $d_{35} = 0.08$ m, $S = 17.0\ m^2$, $O = 13.1$ m, $R = 1.3$ m. The specific flow $q = Q : b'$, b' (central diagonal of trapezoidal profile) $= 8.5$ m, $q = 22.8 : 8.5 = 2.68\ m^3 \cdot s^{-1} \cdot m^{-1}$, $S_d = 9.0\ m^2$, $O_d = 5.0$, $R_d = S_d : O_d = 1.8$ m, $k_d : k_s = 0.8$.

Table 60

Values of coefficient m, depending on the factor of propensity to erosion Z

Z	m	Z	m
0.1	0.5	0.55	0.8
0.2	0.55	0.6	0.85
0.25	0.6	0.7	0.9
0.3	0.65	0.8	1.0
0.4	0.7	0.9	1.05
0.5	0.75	1.0	1.1

Table 61

Values of coefficient φ depending on channel bed gradient i

i (%)	φ	i (%)	φ	i (%)	φ
1.0	0.08	6	0.19	20	0.35
1.5	0.09	7	0.20	25	0.41
1.7	0.10	8	0.22	30	0.49
2.0	0.11	9	0.23	40	0.65
2.5	0.12	10	0.25	50	0.85
3.0	0.13	11	0.26	60	0.97
3.5	0.14	12	0.27	70	1.20
4.0	0.15	13	0.28	80	1.45
4.5	0.16	14	0.30	90	1.75
5.0	0.17	15	0.31	100	2.00

KALINSKE:

$$u_* = \sqrt{Rig} = \sqrt{1.3 \cdot 0.087 \cdot 9.81} = 1.05 \text{ m} \cdot \text{s}^{-1},$$

$$\tau_o = \varrho Rig = 1000 \cdot 1.3 \cdot 0.087 \cdot 9.81 = 1109.51 \text{ Pa},$$

$$\tau_c = 0.06(\varrho_m - \varrho)\, d_m g = 0.06 \cdot (2650 - 1000) \cdot 0.12 \cdot 9.81 = 116.54 \text{ Pa},$$

$$\frac{\tau_c}{\tau_o} = \frac{116.54}{1109.51} = 0.11,$$

$$f\frac{\tau_c}{\tau_o} = f(0.11) \ldots \text{ from Fig. 94. } f(0.11) = 1.8,$$

$$q_{jG} = f(0.11 u_* d_m = 1.8 \cdot 1.05 \cdot 0.12 = 0.227 \text{ m}^3 \cdot \text{s}^{-1} \cdot \text{m}^{-1}.$$

The total volume of bed-load discharge:

$$Q_G = 0.227b = 0.227 \cdot 5.0 = 1.135 \text{ m}^3 \cdot \text{s}^{-1}.$$

The total volume of bed-load discharge during the course of the flood flow:

$$W_G = Q_G t = 1.135 \cdot 2916 = 3310 \text{ m}^3.$$

MEYER-PETER:

$$q_{jG} = \left\{0.86 \cdot \left[\varrho \left(\frac{k_d}{k_s}\right)^{3/2} R_d i - 0.047(\varrho_m - \varrho)\, d_m\right]\right\}^{3/2} =$$

$$= \{0.86 \cdot [1000 \cdot (0.8)^{3/2} \cdot 1.8 \cdot 0.087 - 0.047 \cdot (2650 - 1000) \cdot 0.12]\}^{3/2} =$$

$$= 830 \text{ kg} \cdot \text{s}^{-1} \cdot \text{m}^{-1} \text{ (weight under water)}$$

$$q_{jG} = 830 \cdot \left(\frac{\varrho_m}{\varrho_m - \varrho}\right) = 830 \cdot \left(\frac{2650}{2650 - 1000}\right) = 1333 \text{ kg} \cdot \text{s}^{-1} \cdot \text{m}^{-1},$$

i.e. $0.50 \text{ m}^3 \cdot \text{s}^{-1} \cdot \text{m}^{-1}$,

$$Q_G = 0.50 \cdot 5 = 2.5 \text{ m}^3 \cdot \text{s}^{-1},$$

$$W_G = 2.5 \cdot 2916 = 7290 \text{ m}^3.$$

JÄGGI:

$$q_{jG} = 2.5 \cdot \frac{Q}{b} i^{0.6} \left(i - \frac{d_m}{12.1 h_m}\right) =$$

$$= 2.5 \cdot 4.56 \cdot 0.087^{0.6} \cdot \left(0.087 - \frac{0.12}{12.1 - 2.0}\right) = 0.216 \text{ m}^3 \cdot \text{s}^{-1} \cdot \text{m}^{-1},$$

$$Q_G = 0.216 \cdot 5.0 = 1.08 \text{ m}^3 \cdot \text{s}^{-1},$$

$$W_G = 1.08 \cdot 2916 = 3149 \text{ m}^3.$$

EINSTEIN:

$$\Psi = \frac{\varrho_m - \varrho}{\varrho} \frac{d_{35}}{R_d i} = \frac{2650 - 1000}{1000} \frac{0.08}{1.18 \cdot 0.087} = 1.29.$$

From Fig. 96 for $\Psi = 1.29 \ldots$ $\Phi = 5.5$.

$$q_{jG} = \frac{\Phi \varrho_m}{\sqrt{\dfrac{\varrho}{(\varrho_m - \varrho)\, g d_{35}^3}}} = \frac{5.5 \cdot 2650}{\sqrt{\dfrac{1000}{(2650 - 1000) \cdot 9.81 \cdot 0.08^3}}} =$$

$$= 1326.8 \text{ kg} \cdot \text{s}^{-1} \cdot \text{m}^{-1}, \text{ i.e. } 0.5 \text{ m}^{-3} \cdot \text{s}^{-1} \cdot \text{m}^{-1},$$

$$Q_G = 0.5 \cdot 5.0 = 2.5 \text{ m}^{-3} \cdot \text{s}^{-1},$$

$$W_G = 2.5 \cdot 2916 = 7290 \text{ m}^{-3} \cdot \text{s}^{-1}.$$

b) Sediment material runoff

factor of watershed's susceptibility to erosion:

$Z = XP(E + I^{1/2})$
$X = 0.4$
$P = 0.5$
$E = 0.4$
$I = 25\%$
$Z = 0.4 \cdot 0.5 \cdot (0.4 + 0.25^{1/2}) = 0.16.$

$$Q_G = \frac{2m\varphi Q}{\gamma_m}$$

m for $Z = 0.16$ from Tab. 73 ... $m = 0.52$
φ for $i = 0.087$ from Tab. 74 ... $\varphi = 0.23$

$$Q_G = \frac{2 \cdot 0.52 \cdot 0.23 \cdot 22.8}{2.65} = 2.05 \text{ m}^{-3} \cdot \text{s}^{-1}$$

$W_G = 2.05 \cdot 2916 = 5978 \text{ m}^3$.

The portion of sediment material $W_d = 0.8 \cdot 5978 = 4782 \text{ m}^{-3} \cdot \text{s}^{-1}$.

6.3 Principles of torrent control

The key purpose of torrent control is to control flood flows along torrent channels so as to adequately protect the adjacent buildings, structures, roads, fields and residential areas, to make the torrent channels resistant (both in their direction and depth) to the carrying force of the water current and, in particular, to prevent the rise, transport and deposition of sediments.

6.3.1 Extent of torrent management

The extent of the intervention will primarily depend on the status of the torrent. Systematic torrent management interventions are carried out on torrent channels whose development is incomplete in terms of a course of torrent, cross section and bed slope. Complete systems covering the whole stream from spring to mouth can only be built on short torrents. (Such systems are built in short segments from the source area to the mouth.)

Long torrent streams seldom need training along their whole course. Regulating interventions on such torrents cover long separate segments. In such cases, the torrent must have been investigated in detail to see if the nonregulated upstream parts of the channel would not endanger the regulated segments. If necessary, measures should be taken to remove such dangers. It is particularly important to adequately adjust the place where the nonregulated segment passes into the regulated segment and to prevent the transport of bed-load material to the regulated channel, e.g., by building gravel traps.

It is likewise necessary to assess the effect of the regulated segment on the nonregulated downstream part of the channel in terms of capacity for different flow rates and changed bed-load processes.

Only partial training is designed on torrents having a comparatively stable bed slope but unsuitable flow directions. The management of such torrents includes realignment of the stream's course, stabilisation of local bank scours, stabilisation of gravel bars by building separate sills or weirs, building of gravel traps, etc. Partial regulation is also used where a tributary is bridged and where a parallel road has to be protected at the tributary's mouth.

6.3.2 Methods of torrent control

Torrent control involves a number of specific tasks, including the determination of design discharge, realignment of the stream, adjustment of the bed slope, design of cross section, design of channel lining and design of the structures.

6.3.2.1 Design discharge

Before the technical solutions are sought for torrent control it must first be decided how much protection has to be provided for the land, crops, facilities, residential areas, etc. located within the range of the planned structure's influence.

The level of protection decided on must be sufficient to guarantee that at a certain rate of flow in the regulated segment of the stream the water will not overflow. This "certain rate of flow" is referred to as the design discharge. It is recommended that the determination of the design discharge should be based on a technico-economic analysis of the torrent management measures planned, the costs of construction and normal maintenance of the controlled torrent being compared with the losses caused by floods. It is very difficult to assess the losses at different levels of protection objectively and it should also be taken into account that torrent control brings about a number of various advantages in addition to the economic benefits: considering this, it may be helpful to determine design discharge using the data in Tab. 62.

Table 62

Design flows for channel capacities

Degree of protection	Endangered crops, structures, facilities, etc.	Design flow for channel capacity
1	Compact built-up urban area, larger housing estates, larger villages, industrial plants, important linear structures running parallel (highways, railways etc.), protected monuments	$Q_{50} - Q_{100}$
2	Smaller villages, groups of houses, sporadically built up valleys (distance between homes not less than 100 m), roads of local importance (categories II and III), forest haulage roads, dumps, recreational resorts, fields endangered by gullying.	$Q_{20} - Q_{50}$
3	Outside built-up areas – intensive agricultural production, skidding and other forest roads.	$Q_5 - Q_{20}$
4	Outside built-up areas – meadows, productive forests, irrigation and drainage facilities.	$Q_5 - Q_{10}$

If the torrent-management measures apply to torrent segments both outside and inside villages or towns, or segments passing through land used in different ways (arable, pasture, forest) there will be, as far as technically feasible, different design discharges for channel capacity in the different stream segments.

If such a solution is not feasible or advantageous, a higher discharge is retained as the design discharge also for stream segments in areas where less protection is required.

Torrent streams cause much more damage by devastating their own channels and the property in their vicinity than by floods (Figs 97, 98). Hence, the bottoms and banks of torrents need more protection than the channels of other water courses.

Capacity flow has the greatest influence on the formation of the torrent channel: hence, design discharge for such a torrent bottom's resistance is the same as that for channel capacity. Investigations on torrent channel formation have shown that the depth of water at which the effective grains forming a torrent's bed still do not move corresponds to the depth of a channel formed in a natural way. Hence, flow rates that are higher than the design discharge and cause the inundation of large areas have no significant effects on the formation of the bottom. However, in narrow valleys where the water cannot overflow outside the close banks, the torrent bottom is exposed to a high stress caused by the concentrated flow: in such torrents the bottom's stability has to be assessed in terms of the 100-year water.

Fig.97 Bank scour in a road bed after a flood on the Jílovský brook, Bohemia (Photo P. Vrzák).

Fig.98 Damaged homes after a flood on the Stříbrný brook in the Giant Mts., Bohemia (Photo J. Pacholík).

6.3.2.2 Realignment

Layout principles for torrent-management measures

Correct realignment is among the key prerequisites for maintaining a stable torrent bottom. To protect the banks and the adjacent part of the bottom in concave curves against too much stress during flow in a full channel, the layout of the regulation measures should be as close to the streamline of the design discharge as possible.

This will also meet other requirements, including those for the lowest possible costs of revetment of the banks and for an aesthetic incorporation of the stream in the landscape. When the layout of the regulation measures is designed, smooth and safe flow of water along the channel should not be the only purpose to be considered: account should also be taken of the need to protect the adjacent land, buildings and other property. The alignment should pass through the lowest altitude points of the valley while maintaining the current direction of flow. Maximum use should be made of the original channel so as not to make excessive changes in the land fund, though in some cases it would, on the other hand, be advantageous to re-lay the channel for a better land-fund organisation.

Where a torrent runs parallel to a road or where the road crosses it, wrong alignment may lead to the danger of damage or failure of the road bed. In such places, the alignment has to be drawn with particular care to let the water pass safely under bridges or culverts. It is recommended that roads should cross torrents

perpendicularly or at an angle of 60° at the maximum. Of course, it should be borne in mind that a 90° crossing is suitable where the course of the stream is really perpendicular or near-perpendicular to the road. In other cases, adherence to the principle of perependicular crossing might be dangerous: a forcible adjustment of the stream's direction may endanger not only the stream's channel but also the bed of the road.

The torrent regulation line

The torrent regulation line consists of curves and straight segments which pass smoothly into one another. They key factors most important for optimum sinuosity include correct curvature and length of the bends; adequately long straight or near-straight segments should intervene between successive bends. The routing features should be designed with particular care in torrential streams of the category of small streams which have a narrow ratio of channel width (measured at the water surface) to channel depth and in which simple cross circulation prevails. The designed thalweg must follow smoothly from one curve to another with a minimum shear stress on the banks at the crown of each bend.

The optimum sinuosity has been investigated on natural streams. The analysis of the directions in the natural channels of water courses in Austria proved that neither circular arches nor straight lines are typical of channels which have developed by natural processes. Hence, it is recommended to avoid geometrical arches and straight lines and to use irregular common curves. However, this approach requires much skill and experience in designing the project, and the execution thereof is not easy. Irregular forms can be used with advantage in partial adjustments, e.g., in laying out the bank line when consolidating bank scours or when realigning local sharp bends.

From tachymetric measurements of natural channels, GLÄSER (1968) concluded that the bends have a clotoid shape. NOVÁK (1970) believes that lemniscate is the typical bend form. There are almost no circular bends in naturally developed channels.

Geometrical shapes of applicable alignment bends

The following arc forms can be used in realignment of torrent channels sinuosity: simple circular arc, compound circular arc, clotoid, lemniscate, hyperbola, sometimes also ellipse.

The nature of each arc form will follow from comparing the transition from one bend to another by constructing the curvature line (Fig. 99), the arc length (l) being plotted on axis x and the $l : r$ ratio on axis y.

This graphical survey clearly shows that lemniscate and clotoid are the best arc forms for channel alignment: their curvature line starts on axis x, rising continuously to a maximum point and then successively falling back to axis x. The hyperbola is less suitable: its curvature line starts and ends at a low height above axis x and the transition from one arc to the reverse arc is not so smooth. However, this disadvantage can be resolved by inserting a transition curve, e.g., clotoid. Circular arcs and ellipses are not suitable because their curvature lines do not meet each other, which means that the transition from one bend to another is abrupt and not continuous.

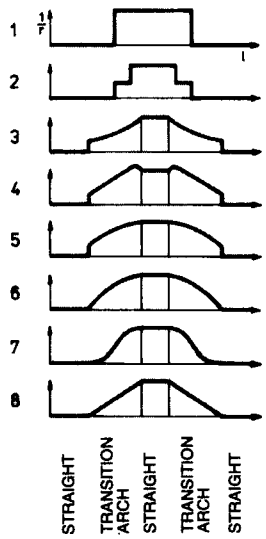


Fig.99 The curvature lines of the geometric forms of bends: 1 – simple circular, 2 – compound circular, 3 – quadratic parabola, 4 – cubic parabola, 5 – hyperbola, 6 – lemniscate, 7 – sinusoid, 8 – clotoid.

Simple circular arcs. As said, circular arcs are not suitable for torrential streams. They may only be used where the central angle is not larger than 20°, and where the channel is well lined. If they are used in ordinary realignment work the following rules have to be respected:

– the radius of the arc on torrents wider than 10 m should be more than 6 B (B is the channel width at the level of design flow) whereas on those less than 10 m wide it should be more than 10 B,

– straight segments $3-4$ B long should be inserted between bends of a high curvature,

– straight segments $2-3$ B long should be inserted between long bends or between those of a low curvature. No straight segments should intervene between bends of a very low curvature: their use may even be harmful.

Compound circular arcs. Compound circular arcs comprise several circular arcs, passing from one into another. Though the curvature of these arcs changes by steps and not smoothly, they are very suitable for adjusting the realignment of torrent channel sinuosity to the natural channel curve. Compound circular arcs also remove the adverse waves which arise in supercritical flow in the bends consisting of simple circular arcs: such waves exert a considerable pressure on the banks and often damage the channel lining and the buildings and facilities on the banks. Compound circular arcs may be either symmetric or asymmetric (Fig. 100). In torrents with a slope $i > 0.01$, the straight segment on the upstream side passes into an arc of a smaller radius and in torrents with a slope $i < 0.01$ it passes into an arc of a greater radius.

For an optimum shape of the compound circular arc, the lengths of the individual arcs are about the same and their radii show the following relationships: $r_1 : r_2 : r_3 = 3 : 2 : 1 = r_1 : r_2 = r_2 : r_3$.

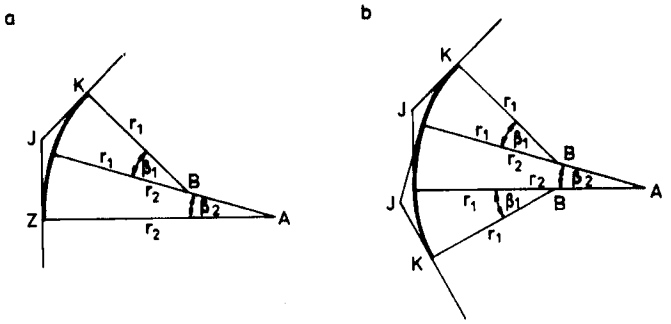


Fig.100 Compound circular bend: a – asymmetrical, b – symmetrical.

Clotoid arcs. The clotoid is a spiral for which it holds that the product of the length l (from zero to a given point) and the diameter at that point is constant (Fig. 101):

$$C = rl = A^2, \tag{6.64}$$

where A – parameter of the clotoid.

In this type of bend, the radius decreases continuously and curvature increases with increasing length. This form can be used to attain a channel line best corresponding to the thalweg.

Lemniscate arcs. The lemniscate is a curve for which it holds that the product of the length of a chord s from zero to a given point and the radius at that point is constant (Fig. 102):

$$C = rs. \tag{6.65}$$

Thus, its curvature continuously increases; hence, like the clotoid, the lemniscate is suitable for torrent realignment. Its shape is the same as that of the clotoid up to a tangent angle of 100^g.

Lemniscate arcs designed for torrent channel alignment may be both symmetric and asymmetric (Fig. 103). When assessing the directional characteristics of the curved parts of natural torrents, both types of lemniscate arcs were found to occur under natural conditions. A longer tangent (i.e., smaller curvature) connects the asymmetric lemniscate arc with the upstream part of the realigned segment of torrents with a bottom slope $i < 1\%$ and a shorter tangent (i.e., greater curvature) does so in torrents with a bottom slope $i > 1\%$.

The minimum lengths of lemniscate arcs depend on the $B : h$ ratio. The smaller this ratio (the narrower and deeper the channel) the longer the lemniscate arc required. No straight segments are needed between the lemniscate bends: they are primarily designed for the cases in which asymmetric lemniscates are used on the side with the shorter tangent, i.e., with a higher curvature. The maximum length of the straight segment depends on the $B : h$ ratio: the narrower this ratio the shorter the straight part of the line.

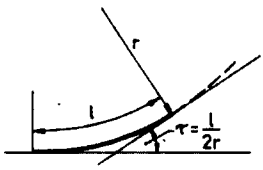


Fig.101 Clotoid.

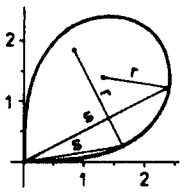


Fig.102 Lemniscate.

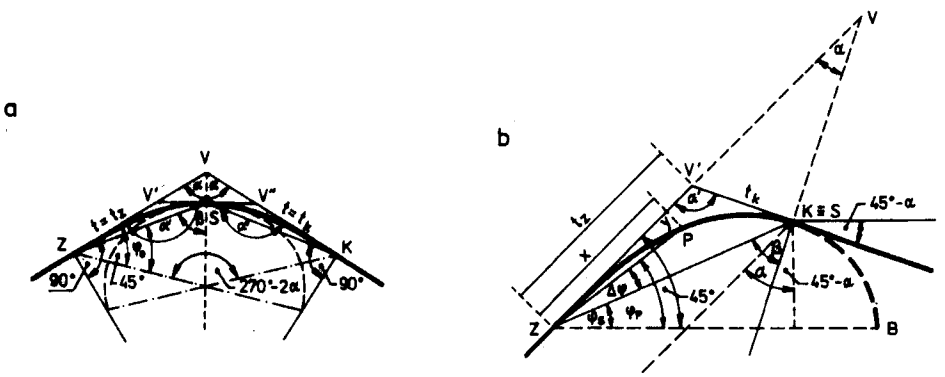


Fig.103 Lemniscate bend: a – symmetrical, b – asymmetrical.

6.3.2.3 Adjustment of the bed slope

The stability of torrential stream bottoms, i.e. equilibrium between the carrying force of the water stream and the resistance of the bottom material against being raised and transported, can be achieved either by reducing the kinetic energy (through adjusting the bed slope) or by an increased roughness of the channel, or by a combination of both.

A reduction of kinetic energy can also be attained by widening the bottom, hence by reducing the depth of the design discharge. However, this method cannot be widely used because with fluctuating flow in a wide channel, small particles of eroded material carried by the water might settle on the bottom if the flow rates were less than the design discharge. At higher flow rates, the sediment may lead to undesirable hydraulic changes and cause damage in the channel. Hence, the slope of the bottom should be designed in compliance with the design of the cross section.

Everywhere possible the torrent bottom should be left unprotected, lining being resorted to only in stringent circumstances in built-up areas. Of course, the design for bed slope adjustment must be subjected to economic analysis.

Adjusting the slope of nonlined bottoms

The nonlined torrent bottom is created of sediment mixture. The objective of adjusting the slope is to prevent the scouring of sediment and the deposition of the bed load carried by water from the upstream parts of the torrent, or to ensure that

within the given profile the amount of incoming material is the same as the amount of material moving further downstream, i.e., that the level of the bottom remains unchanged.

The slope at which such undisturbed flow of bed load is maintained is referred to as the equilibrium bed slope. To form an equilibrium slope is a difficult and complex task, depending not only on sediment characteristics but also on the flow of bottom-load material and on the amount of such material carried by the water.

At a high water saturation with bed load and wash, the water expends much energy in the transport of the material and is able to lift only small particles from the bottom (or it leaves the bottom intact). If the bed slope is less than a certain threshold value, the energy of the water is too low to carry the material and it drops onto the bottom. This may explain why torrent channels are clogged with gravel in places where the bottom was adjusted to a stable bed slope. On the other hand, an equilibrium bed slope adjusted to a maximum flow of bed load and wash may be damaged by the flow if the water is "clean" or carries only a small amount of material. This happens when sediment is deposited in the channel or above check dams.

Determining a stable bed slope. Two theories find use in determining the stable bed slope: the theory of incipient sediment movement and the theory of bed-load movement. Data on critical bed velocities, critical average channel velocities, critical shear stress and intensity of bed-load movement are the basic data used in the calculation.

Method of critical bed velocities. Sediment particles (diameter d_m) will remain stable in the surface layer of the bottom provided that the actual bed velocity is less than, or equal to, the critical bed velocity.

Bed velocity can be determined from the logarithmic velocity distribution after NIKURADZE, using the equation adjusted by KEULEGAN (1938):

$$u_d = \left(5.75 \cdot \log \frac{30y}{k}\right) v_* \qquad (\mathrm{m \cdot s^{-1}}), \tag{6.66}$$

where y – depth of water at the point of origin of vertical velocity (m),
k – size of the projections forming the coarseness of the bottom (m),
v_* – shear velocity $(\mathrm{m \cdot s^{-1}})$; $v_* = \sqrt{Rig}$.

EINSTEIN and EL SAMNI (1949) found that the velocity near the bottom which influences the stability of the sediment particles originates at a depth of $0.35-0.77k$ above the ideal bottom. In torrential streams with a significantly uneven bottom, the bed velocity can be assumed to originate at a depth of $y = 0.35d_m$.

Bottom roughness is well characterised by the parameters of bottom projections. The key bottom-roughness parameter is the mean value of bottom projection height which can be determined from the mean arithmetic deviation from the median line of the profile

$$R_a = \frac{1}{n} \sum_{i=1}^{n} |y_i| \qquad (\mathrm{m}), \tag{6.67}$$

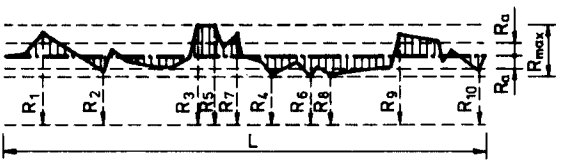


Fig.104 Geometrical characteristics of profile roughness.

where n – number of the measured deviations from the median line,
y_i – distance from the ith point of the unevenness of the bottom profile to the median line (Fig. 104).

Since the mean arithmetic deviation expresses the mean distance of the projections both above and below the median line of the profile, the average height of the projections will be

$$k = 2R_a \qquad \text{(m)}. \tag{6.68}$$

NOVÁK (1987), measuring the very rough channels of torrents in the Giant Mountains, found that R_a depends on the standard grain of the surface layer of the bottom:

$$R_a = 0.603 d_m \qquad \text{(m)}. \tag{6.69}$$

Then

$$k = 2R_a = 1.206 d_m \qquad \text{(m)}. \tag{6.70}$$

Inserting the values of $y = 0.35 d_m$, $k = 1.206 d_m$ and $v_* = \sqrt{Rig}$ in equation (6.66) will produce the equation for bed velocity:

$$u_d = \left(5.75 \cdot \log 30 \cdot \frac{0.35 d_m}{1.206 d_m}\right) \sqrt{Rig} \qquad (\text{m} \cdot \text{s}^{-1}) \tag{6.71}$$

and after rearrangement the equation will be

$$u_d = 5.4 \sqrt{Rig} \qquad (\text{m} \cdot \text{s}^{-1}). \tag{6.72}$$

Comparison of the equation for the bed velocity (6.72) with the equation for the critical bed velocity (6.25)

$$5.4 \sqrt{Rig} = C d_m^{0.5} \tag{6.73}$$

will allow us to attain the equation for a stable bed slope ($i = i_s$):

$$i_s = 0.0035 C^2 \frac{d_m}{R}. \tag{6.74}$$

Using the average values of constant C, the equation for a stable bed slope can be adjusted to the following form:

$$i_s = 0.101 \frac{d_m}{R} \tag{6.75}$$

for rounded sediment particles ($C = 5.38)R$ and

$$i_s = 0.074 \frac{d_m}{R} \tag{6.76}$$

for angular particles ($C = 4.59$).

VALENTINI (1895) used the equation for a critical bed velocity to derive the equation for a stable bed slope: he assumed that

$$\left(\frac{d_m(\varrho_m - \varrho) f \cdot 2g}{0.076\varrho}\right)^{0.5} = C\sqrt{Ri}\,. \tag{6.77}$$

From this he derived the equation for a stable bed slope

$$i_s = \frac{d_m(\varrho_m - \varrho) f \cdot 2g}{0.076\varrho C^2 R}\,. \tag{6.78}$$

Having expressed the constants in numerical values, he rearranged the equation:

$$i_s = 0.093 \frac{d_m}{R}\,. \tag{6.79}$$

Method of critical average channel velocities. An equation for the stable bed slope can also be obtained by comparing the equation for the cross-sectional velocity with that for the critical average channel velocity after SRIBNYI (1932) (6.36) or after NEILL (1967) (6.34):

SRIBNYI:

$$kR^{2/3} i^{1/2} = 5 d_m^{0.5} \tag{6.80}$$

$$i_s = \frac{25 d_m}{k^2 R^{4/3}}, \tag{6.81}$$

where $k = \frac{1}{n}$,

n – degree of roughness to be found in Tab. 63.

Table 63

Degrees of roughness of mountain and periodic water courses (after SRIBNYI)

Stream channel characteristics	n	$\frac{1}{n}$
Earth channels of periodic water courses in favourable environments. Well developed gravelly channels of mountain water courses. $i = 0.003-0.007$	0.040	25
Periodic water courses, moving significant amount of sediment in flood periods, beds filled with crude gravel or covered with plants. Channels of medium and large water courses dirty, partly overgrown, stony. $i = 0.007-0.015$	0.050	20
Channels of periodic water courses, very dirty, zigzagging. Mountain-type channels filled with gravel and pebbles. $i = 0.015-0.05$	0.067	15
Bouldery channels of mountain streams. $i = 0.08-0.09$	0.080	12.5
Bouldery channels of mountain streams, river beds of rough boulders, zigzagging. $i = 0.09-0.20$	0.100	10

NEILL:

$$kR^{2/3}i^{1/2} = \left[2.5\left(\frac{h}{d_m}\right)^{0.2} g\left(\frac{\varrho_m - \varrho}{\varrho}\right) d_m\right]^{0.5}, \tag{6.82}$$

$$i_s = \frac{2.5\left(\frac{h}{d_m}\right)^{0.2} g\left(\frac{\varrho_m - \varrho}{\varrho}\right) d_m}{k^2 R^{4/3}}, \tag{6.83}$$

where $k = \frac{a}{d_m^{1/6}}$,

a – constant (see section 6.3.2.4).

The stable bed slope can also be derived from empirically determined values of critical average channel velocities (e.g., those in the former Soviet Standard St-24-2396), using the equation

$$i_s = \frac{v_v^2}{k^2 R^{4/3}}, \tag{6.84}$$

where v_v – critical average channel velocity determined from the nomogram in Fig. 91: it will depend on the size of the standard grain and on the depth of water in the channel ($m \cdot s^{-1}$).

Method of critical shear stress. Stability of the sediment mix is attained if the actual shear stress is lower than or equal to the critical shear stress. The equations for critical shear stress (6.37 and 6.38) can be used to calculate the stable bed slope:

SHIELDS (1936):

$$\varrho Rig = 0.06(\varrho_m - \varrho)\, d_m g \tag{6.85}$$

$$i_s = \frac{0.06(\varrho_m - \varrho)\, d_m}{\varrho R}. \tag{6.86}$$

EGIAZAROV (1965):

$$\varrho Rig = \frac{0.1(\varrho_m - \varrho)\, d_{50} g}{\left(\log 19 \frac{d_{50}}{d_m}\right)^2}. \tag{6.87}$$

$$i_s = \frac{0.1(\varrho_m - \varrho)\, d_{50}}{\left(\log 19 \frac{d_{50}}{d_m}\right) \varrho R}. \tag{6.88}$$

Method of bed load movement intensity. Though the equations for bed-load movement are not limited by critical values, they can be used for determining the incipient movement and thereby for determining the stable bed slope.

MEYER-PETER (1949):

Providing that there is no bed-load movement in the channel ($q_{jG} = O$), equation (6.41) will express the relationship between the actual and critical shear stress from which the equation for a stable bed slope can be derived:

$$\varrho R_d \left(\frac{k_d}{k_s}\right)^{3/2} i = 0.047(\varrho_m - \varrho)\, d_m \tag{6.89}$$

$$i_s = \frac{0.047(\varrho_m - \varrho)\, d_m}{\varrho R_d \left(\frac{k_d}{k_s}\right)^{3/2}}.$$

EINSTEIN (1950):

Equation (6.57) for flow intensity can be rearranged to express the relationship between the actual and critical shear stress:

$$\varrho R_d i = \frac{1}{\Psi}(\varrho_m - \varrho)\, d_m. \tag{6.91}$$

The stable bed slope can be determined from this equation:

$$i_s = \frac{\frac{1}{\Psi}(\varrho_m - \varrho)\, d_m}{\varrho R_d}. \tag{6.92}$$

According to GRAF (1971), the critical value of $1/\Psi$ for the incipient movement of the bed-load grains is $1/\Psi = 0.056$.

Designing the bed slope. Sills, weirs and chutes are built into torrents to remove the difference between the original and designed slope. The level line of the bottom is designed so that the construction should require minimum earthwork to build the adjusted torrent channel in the original terrain and no levels to line the channel.

The level line of the bed should not affect the optimum groundwater level in the adjacent areas and should provide optimum conditions for the tributaries, canals and drainage systems to empty their waters into the torrent.

When fitting the level line into the longitudinal profile, the depth of the designed cross section should be adhered to and the level line should connect each weir crest with the toe of the upstream weir. Depending on the depth of the original channel, the following cases may occur:

– in channels that are very deep along their whole length, the bottom will have to be raised: the level line will be above the level of the present bottom (Fig. 105),

– in channels that are unable to accommodate the flow capacity and in new segments outside the original channel, the new channel will have to be deeper and the level line will be below the current level of the bed or ground surface (Fig. 106),

– in channels with an irregular slope where shallow segments pass into segments with a deeper channel bed or where a deep original channel passes into a shallow ditch cut through a meander, the level line will fluctuate up and down from the level of the original bed or ground surface (Fig. 107).

The location of transverse structures (dams, sills, weirs) is designed with respect to the current conditions of the water course being regulated. The structures do not need to be uniformly spaced or have the same height: in fact, uniform distances and heights may even cause difficulties. Transverse structures should be located where the level line is too deep under the current level of the channel bed or ground surface or under the mouth of the ditch cut through a meander, or under the mouth

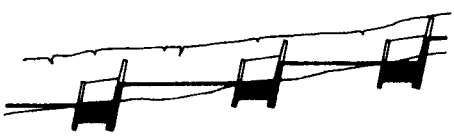


Fig.105 Adjustment of the channel-bed slope in a very deep channel.

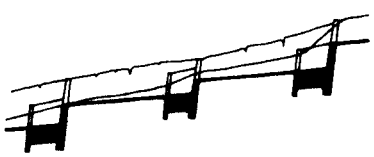


Fig.106 Adjustment of the channel-bed slope in a shallow channel.

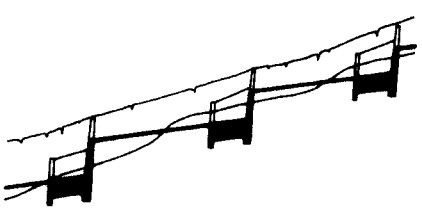


Fig.107 Adjustment of the channel-bed slope in a channel of irregular depth.

of a tributary, and everywhere there are large altitude differences in the current channel bed.

When designing the level line of the channel bed, it must also be decided how high the transverse structures are to be. The main factors on which the height of such structures depends include the local conditions of the water course, economic considerations and the need to maintain the natural bed-load processes.

JAŘABÁČ (1972) believes that the gradient of torrents carrying large amounts of gravel can be adjusted with advantage using simple low sills ("Silesian type" of torrent management): this approach is inexpensive and does not affect the natural flow of bed load. The adjusted bottom is comparatively shallow.

Adjusting the slope of lined channel bottom

Stepping of unlined channel bottom changes the natural characteristics of the torrent and is costly. It is also uneconomical to build weirs in torrents having a very steep slope: too many weirs would be required to reach a stable slope in such cases and the weirs would have to be high. It is more advantageous only to level out local unevennesses and to line the bottom, maintaining the original slope. Two approaches are available: either to use materials not affecting the natural look of the torrent (nature-like approach) or to apply a structural solution.

Nature-like channel bottom lining. This approach comprises two options, either lining the bottom with a rock riprap or improving the bottom's resistance using scattered boulders.

Rock riprap

A layer of nongraded gravel will improve the resistance of the bottom while maintaining a stable bed slope not interrupted, as far as possible, by weirs. The general objective in fact is to attain a maximum possible stability of the bed slope. The gravel used to line the bottom should be nongraded and adequately structured. The effective grain of the added gravel layer should be of optimum size to prevent damage to the bottom at the given slope. The optimum effective grain size can be calculated using the equations for stable gradients of nonlined bed channels, e.g.:

NOVÁK:

$$d_m = \frac{i_s R}{0.0035 C^2}, \tag{6.93}$$

VALENTINI:

$$d_m = \frac{i_s R}{0.093}, \tag{6.94}$$

SRIBNYI:

$$d_m = \frac{i_s k^2 R^{4/3}}{25}, \tag{6.95}$$

SHIELDS:

$$d_m = \frac{i_s \varrho R}{0.06(\varrho_m - \varrho)}, \tag{6.96}$$

MEYER-PETER:

$$d_m = \frac{i_s \varrho R_d \left(\frac{k_d}{k_s}\right)^{3/2}}{0.047(\varrho_m - \varrho)}, \tag{6.97}$$

EINSTEIN:

$$d_m = \frac{i_s \varrho R_d}{0.056(\varrho_m - \varrho)}, \tag{6.98}$$

Improving the bottom's resistance using scattered boulders

On torrential streams having a more or less homogeneous cover layer on the bottom, a higher channel bed resistance can be achieved by distributing large boulders whose dimensions are much larger than those of the grains composing the cover layer (Fig. 108). At higher flow rates, the boulders are undermined on their downstream side: thus they progressively sink deeper into the cover layer of the bottom where they may act as a stabilising factor. They also increase the roughness of the bottom, thus reducing the average channel velocity. This approach was used with success, for example, on the Lomná torrent on the Czech side of the Beskid Mountains.

It is very difficult to determine the optimum boulder size and distribution pattern. Nevertheless, such a determination is important for the smooth passage of bed load without changing the channel bed level. If the boulders are small and their distribution thin it may happen that the stress τ_d exerted on the bottom as a whole will be too small so that the separate grains of the bed load will share an excessively large portion of the total stress $\tau_o (\tau_s = \tau_o = \tau_d)$: sediment will thus begin to be lifted from the cover layer of the bottom and if this material moves en masse the bottom deepens. On the other hand, if the boulders are too big and dense, the water flow energy may be reduced too much and the bottom load brought from the upstream segments of the torrent may begin to settle in the treated segment of the stream.

Structural solution for channel bed lining. The most frequent method in the structural approach is the use of paving stone laid either directly on the bottom or into cement mortar or onto a concrete bed; lining with concrete blocks is less frequent. In such cases a stable bed slope is designed on the basis of permitted average channel velocities (see Tab. 65).

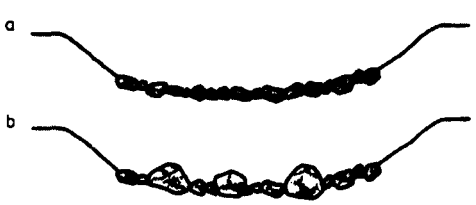


Fig.108 Improving bottom resistance using large boulders: a – before the intervention, b – after the intervention.

When designing the channel bed slope, the maximum admissible average channel velocities can be used with advantage (without stepping the bottom, as far as possible). If the slope is to be reduced to allow only low water velocities to develop in the channel, weirs should be built; however, this not only increases the costs but may also lead to devastation of the channel. Higher slopes of lined channels cause supercritical flows which in turn cause an excessive speed of the nappe falling from the spillway; the apron, however strong its lining may be, cannot resist the nappe's energy. As a result scours develop in the apron and the lining material may be removed by the water from the adjacent parts of the channel. Stabilising belts may be built in the channel (Fig. 109) to improve the stability of the lined bottom or to confine damage to smaller segments.

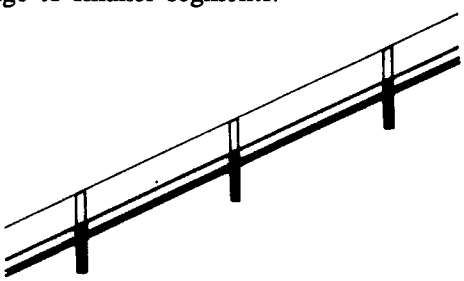


Fig.109 Division of a lined bottom by stabilising belts.

6.3.2.4 Designing the channel cross section

The cross section of a torrent should convey the design discharge without damaging the banks. The design work includes decisions on cross section shape, dimensions and lining. Open profiles should be preferred in torrents: a closed profile may be of help in exceptional cases (e.g., within residential areas) but there is always a danger that objects carried down by the water at flood flows may clog the inlet of the closed profile and the water may then spread outside the bed, causing damage by leaving sediment in the surrounding area.

Cross section shape

The key factors on which the design of cross section shape depends include the discharge and local conditions. The choice of the shape is limited inside villages or towns, in places where the torrent and a road run parallel or where the torrent passes other facilities or structures, and in closed narrow valleys. On the other hand, outside built-up areas in forests or fields there is enough room to choose the optimum shape of cross section. The factors to be taken into account in the design also include the general possibilities of the intervention, the method of bank revetment and the possibilities of maintenance.

Single trapezoidal cross section

The single trapezoidal cross section (Fig. 110) is the most common torrent channel shape. It is particularly suitable for narrow channels of torrents in which there is no great difference between the depth of the actual steady and the design discharges. The bed load transported by torrential streams at increased flow rates

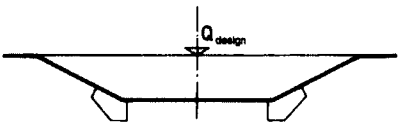


Fig.110 Single trapezoidal cross-section.

largely consists of gravel, so torrent channels are not so much silted up with fine particles as are the beds of other streams and rivers. Nevertheless, it is necessary, mainly in segments running through farmland in submontaneous areas, that the width of this type of bed should be adjusted to the required water depth: at a 210-day flow rate the depth should be 0.2 m at the minimum and the velocity of the flow should not decrease below 0.4 $m \cdot s^{-1}$.

The physical properties of the earth forming the banks (especially the natural angle of slope) should be taken into account when designing the gradient of the slopes; the type of bank revetment and the space available for the formation of the channel are also important. In a built-up area where only limited space is available, the gradient of the slopes is 5 : 1 (banks lined by embankment walls) but where more space is available it can be 1 : 1 to 1 : 1.5 (banks lined by pavement).

Outside built-up areas, in torrent segments running through grassland, arable and forest land, the gradient of the slopes is 1 : 1.5 to 1 : 3. The majority of the existing revetment features can be used at such gradients. With a single trapezoidal cross section, the banks being lined up with a lath fence, there is a rectangular channel in the bottom part which can safely convey a 30-day flow.

Double trapezoidal cross section

This cross section shape is suitable for larger torrential streams with considerably fluctuating flow rates and with mass movement of bottom load. The double trapezoidal cross section consists of two trapezoidal sections, one on top of the other (Fig. 111). The lower section, called the cunette, is designed to convey the annual flood and the upper profile accommodates excess water when the flow is higher than the annual maximum. The upper section's floor, called the berm, is designed at a gradient of 1 : 10.

Dished cross section

The dished cross section is the closest to the natural shape of torrent channels. Though the channel may be wide, its recessed middle part provides sufficient depth for a 210-day flow. However, this profile concentrates the flow in the middle of the channel where it produces a considerable shear stress, hence, it is only suitable for torrents whose bottom is paved or lined with a layer of rock riprap.

Cross section dimensions

The size of the channel should be sufficient to safely convey the design discharge. In torrent control, the discharge is always associated with the determination of a stable channel-bed slope gradient.

Assuming that there is a uniform, steady flow in the regular channel, the cross section dimensions should statisfy the following condition:

$$Q = Sv = Q_n \qquad (m^3 \cdot s^{-1}), \tag{6.99}$$

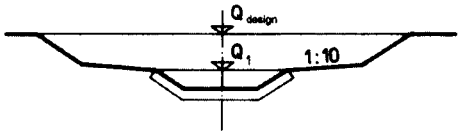


Fig.111 Double trapezoidal cross-section.

where Q – discharge ($m^3 \cdot s^{-1}$),
S – cross section area (m^2),
v – average channel velocity ($m \cdot s^{-1}$),
Q_n – design discharge ($m^3 \cdot s^{-1}$).

The average channel velocity can be determined from the formulae derived by the following authors:

CHÉZY: $$v = C\sqrt{Ri} \qquad (m \cdot s^{-1}), \tag{6.100}$$

STRICKLER: $$v = kR^{2/3}i^{1/2} \qquad (m \cdot s^{-1}), \tag{6.101}$$

where C – coefficient of velocity,
R – hydraulic radius (m)
i – energy line slope,
k – coefficient of velocity.

The coefficient of velocity comprises the losses caused by friction. Correct determination of this coefficient is necessary for a correct determination of the average channel velocity and hence also the discharge. The value of the coefficient depends on the roughness of the whole channel, comprising the roughness of the channel as a whole, of the sediment and of the lining (both in the channel bottom and on the banks), as well as the size and shape of cross section, the curvature of the channel and other factors: their effect is expressed by the roughness coefficient n (Tab. 64).

The value of the velocity coefficient C for very rough torrent channels can be calculated from formulas proposed by the following three authors:

PAVLOVSKY: $$C = \frac{1}{n} R^y \tag{6.102}$$

$$y = 2.5\sqrt{n} - 0.13 - 0.75\sqrt{R}(\sqrt{n} - 0.10), \tag{6.103}$$

MANNING: $$C = \frac{1}{n} R^{1/6} \tag{6.104}$$

LIBÝ (1977): $$C = 17.72 \log \frac{1.85\, R_t}{R_a}, \tag{6.105}$$

Table 64

Degrees of roughness for different types of channel linings and for natural torrent channels

Material	Construction	n		
		min.	medium	max.
Concrete	smooth cement plaster		0.011	
	smoothed concrete	0.011	0.013	0.015
	concrete blocks, concrete tubes	0.014	0.017	0.020
	rough concrete, rough surface	0.016	0.019	0.023
	concrete cavity blocks with cavities overgrown by grass	0.023	0.028	0.038
	concrete bottom with walls of quarry stone & cement mortar	0.020	0.025	0.030
	the same with dry masonry of quarry stone	0.024	0.030	0.036
	gravelly bottom with walls of concrete or solid prefabricated concrete parts	0.017	0.020	0.025
Paving	paving of quarry stone & cement mortar, jointed	0.017	0.024	0.030
	paving of large dug stones & cement mortar	0.020	0.023	0.026
	paving of dry dug stones	0.023	0.032	0.035
Masonry	masonry of worked stone	0.013	0.015	0.017
	masonry of quarry stone & mortar	0.017	0.025	0.030
	dry masonry of quarry stone, stone filling	0.023	0.032	0.035
Lath fence	gravelly bottom, riprap			
	d_m 10 cm		0.025	
	d_m 10 – 15 cm		0.033	
	d_m 15 – 20 cm		0.035	
	Brushwood "carpet" on bottom		0.035	
Vegetated structures	3-row log dams	0.035	0.045	0.055
	stone packing	0.040	0.050	0.060
Vegetation lining	**grass cover:**			
	short, well maintained	0.025	0.030	0.035
	tall, weedy	0.030	0.035	0.050
	shrub stands, willows:			
	thin, with weeds	0.035	0.050	0.060
	medium-dense, lopped	0.040	0.060	0.080
	very dense (of willow brushwood covering) not maintained	0.070	0.100	0.140
	tree stands:			
	1 – 2-row riparian stand in berm above cunette, well maintained, no undergrowth	0.035	0.050	0.075
	irregular riparian stand along whole length of bank slope, with coppices and undergrowth	0.080	0.100	0.120
Natural torrents with gravelly bed and with sporadically damaged bank, overgrown with grass, in some places with shrubs		0.035	0.045	0.055
Mountain torrents, gravelly bed with boulders, banks steep and irregular, overgrown with shrubs and grass		0.040	0.050	0.060
Bottom stony to bouldery, banks overgrown with shrubs and trees		0.050	0.060	0.070

where n – coefficient of roughness,
R_t – substitute hydraulic radius (m):
for narrow trapezoidal cross section $R_t = 0.9R$,
for wide trapezoidal cross section $R_t = 0.8R$,
R_a – mean arithmetic deviation of the projections responsible for the roughnesss (m).

The coefficient of velocity k in Strickler's equation (6.101) for the bottom is determined from the formula

$$k = \frac{1}{n} = \frac{a}{d_m^{1/6}}, \tag{6.106}$$

where a – constant expressing the ratio of the effective grain to the degree of roughness.

STRICKLER (1923) determined constant a as 21.1 for sandy bottoms. PIRKOVSKÝ (1965) found that for the relative smoothness of $R/d_m > 5$ the best value of constant a is 19.4 and for a great roughness the best value is 15. NOVÁK (1987) analysed measurements from bottoms consisting of sediment at a d_m of 0.013 – 0.2 m: he found that constant a varies and (depending on the effective grain) can be expressed by the following relationship:

$$a = 20.95 - 0.459\, d_m + 0.000417\, d_m^3. \tag{6.107}$$

GLADKI (1976) performed measurements in torrents in the Carpathians and used the results of his measurements to derive the formula for the velocity coefficient k:

$$k = 54.58\, d_m^{4.53}\, d_m \tag{6.108}$$

KALIŠ (1983) subjected a layer of gravel (diameter 0.07 – 0.15 m) to laboratory measurement and obtained a value of 20.68 for constant a; for practical use he recommends to reduce this value by 10 %, i.e. $a = 18.6$.

Because the roughness of the bottom varies in the given profile with the material forming the bottom and the banks, the determined partial values of the degree of roughness should be converted to an average value applying to the whole wetted perimeter. This can be done either by the methods developed by KALIŠ or by the method after FORCHHEIMER:

KALIŠ
$$n = (\sum a_i n_i^{3/2})^{2/3}, \tag{6.109}$$

where $a_i = o_i/o$
o – wetted perimeter of the whole cross section
o_i – part of the wetted perimeter whose roughness is n_i.

FORCHHEIMER:
$$n = \left(\frac{\sum O_i n_i^2}{o}\right)^{1/2}. \tag{6.110}$$

Example 3

Design the channel's cross section and determine the stable channel-bed slope under the following conditions: $Q_n = Q_{10} = 27.0\ m^3 \cdot s^{-1}$, $d_m = 0.09$ m, $d_{50} = 0.08$ m; constant from equation (6.74) $C = 5.38$.

Designing the cross section

The shape and lining of the flow profile to be designed are as shown in Fig. 112. The depth h is to be 1.2 m. Because the discharge in the profile has to be evaluated together with the determination of the stable channel-bed slope, the first parameter to be determined for the preliminary design is the cross section area needed to convey the design discharge. The critical average channel velocity v_v is used as a basis: the monogram in Fig. 91 will show that for the chosen depth $h = 1.2$ m and $d_m = 0.09$ the v_v will be 2.72 $m \cdot s^{-1}$. The required cross section area then will be

$$S = \frac{Q}{v_v} = \frac{27.0}{2.72} = 9.92\ m^2.$$

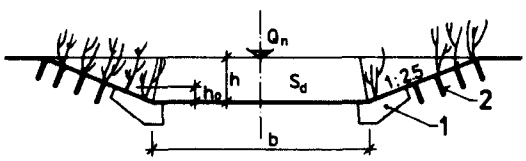


Fig.112 Design of cross-section for Example 3.

The corresponding bottom width will be determined from the equation for the area of trapezoidal cross section:

$$b = \frac{S - mh^2}{h} = \frac{9.92 - 2.5 \cdot 1.2^2}{1.2} = 5.67\ m \approx 6.0\ m.$$

Determining the stable channel-bed slope

The basic hydraulic characteristics of the profile designed include $b = 6.0$ m, $h = 1.2$ m, $S = 10.8\ m^2$, $O = 12.46$ m, $R = 0.867$. The velocity coefficient k in the Strickler equation for the whole cross section is determined from the equation

$$k = \left(\frac{o_1 k_1^2 + o_2 k_2^2}{o}\right)^{0.5},$$

where o_1 = part of wetted perimeter due to the bottom; $o_1 = 6.0$ m,

o_2 – part of wetted perimeter due to the channel slopes which are lined with shrubs (willows): $o_2 = 6.46$ m,

k_1 – velocity coefficient appertaining to the bottom:

$$k_1 = \frac{a}{d_m^{1/6}} = \frac{17.12}{0.09^{1/6}} = 25.57.$$

k_2 = velocity coefficient appertaining to the willow stand whose degree of roughness (Tab. 64) $n_2 = 0.040$:

$$k_2 = \frac{1}{n_2} = \frac{1}{0.040} = 25$$

$$k = \left(\frac{6.0 \cdot 25.57^2 + 6.46 \cdot 25^2}{12.46}\right)^{0.5} = 25.27.$$

According to different authors, the stable channel-bed slope is as follows:

VALENTINI: $$i_s = 0.093 \frac{d_m}{R} = 0.093 \frac{0.09}{0.87} = 0.0096$$

NOVÁK:
$$i_s = 0{,}0035 C^2 \frac{d_m}{R} = 0.0035 \cdot 5.38^2 \frac{0.09}{0.87} = 0.0105 .$$

NEILL:
$$i_s = \frac{2.5 \left(\frac{h^{0.2}}{d_m}\right) g \left(\frac{\varrho m - \varrho}{\varrho}\right) d_m}{k^2 R^{4/3}} =$$

$$= \frac{2.5 \cdot \left(\frac{1.2}{0.09}\right)^{0.2} \cdot 9.81 \cdot \frac{2680 - 1000}{1000} \cdot 0.09}{25.27^2 \cdot 0.87^{4/3}} = 0.012 ,$$

using the critical average channel velocity:

$$i_s = \frac{v_v^2}{k^2 R^{4/3}} = \frac{2.72^2}{25.27^2 \cdot 0.87^{4/3}} = 0.014 .$$

SHIELDS:
$$i_s = \frac{0.06(\varrho_m - \varrho)\, d_m}{\varrho R} = \frac{0.06 \cdot (2680 - 1000) \cdot 0.09}{1000 \cdot 0.87^{4/3}} = 0.0104 .$$

EGIAZAROV:
$$i_s = \frac{0.1(\varrho_m - \varrho)\, d_{50}}{\left(\log 19 \frac{d_{50}}{d_m}\right)^2 \varrho R} = \frac{0.1 \cdot (2680 - 1000) \cdot 0.08}{\left(\log \frac{0.08}{0.09}\right)^2 \cdot 1000 \cdot 0.87} = 0.0103 .$$

MEYER-PETER:
$$i_s = \frac{0.047(\varrho_m - \varrho)\, d_m}{\varrho R_d \left(\frac{k_d}{k_s}\right)^{3/2}}$$

$$R_d = \frac{S_d}{O_d} = \frac{6.9}{6.0} = 1.15 ,$$

$$\left(\frac{k_d}{k_s}\right)^{3/2} = 0.8^{3/2} = 0.72$$

$$i_s = \frac{0.047 \cdot (2680 - 1000) \cdot 0.09}{1.15 \cdot 0.72 \cdot 1000} = 0.0086 .$$

EINSTEIN:
$$i_s = \frac{0.056(\varrho_m - \varrho)\, d_m}{\varrho R_d} = \frac{0.056 \cdot (2680 - 1000) \cdot 0.09}{1000 \cdot 1.15} = 0.0074 .$$

The value of $i_s = 0.011$ is chosen from the calculated values of the stable channel-bed slope. Assessing the discharge in the cross section,

$$Q = Sv$$

$$v = kR^{2/3} i^{1/2} = 25.27 \cdot 0.87^{2/3} \cdot 0.011^{1/2} = 2.42 \text{ m} \cdot \text{s}^{-1}$$

$$Q = 10.8 \cdot 2.42 = 26.14 \text{ m}^3 \cdot \text{s}^{-1} \approx 27.0 \text{ m}^3 \cdot \text{s}^{-1} .$$

6.3.2.5 Channel lining

Channel lining includes the lining of the bottom and of the side slopes. The decision as to which type of lining to choose depends on the critical velocity corresponding to each particular type of lining (Tab. 65): this velocity must be higher than the average channel velocity at design discharge. The reduction of the critical velocity in the channel parts exposed to the greatest stress should be taken into account when considering the type and parameters of lining for such parts (parts of the bottom on the concave side of bend, the toe of side slopes, especially on the concave side in bend).

Table 65

Admissible average channel velocities for lined slopes

Type of lining		admissible v_s ($m \cdot s^{-1}$) at medium depth (m)		
		0.4	1.0	2.0
Sowing		0.6	0.8	1.0
Sodding	whole-area frontal	1.0 1.5	1.5 1.8	2.0 2.2
Willow brushwood covering		2.0	2.2	2.5
Riprap Riprap among wicker fences	min. thickness 30 cm	– –	4.0 4.7	4.5 5.5
Stone pitching	thickness 20 cm	2.5	3.0	3.5
Stone packing, stone masonry	min. strength 9.8 MPa 29.4 MPa	 3.0 6.5	 3.5 8.0	 4.0 10.0
Stone-filling gabion	thickness 30 cm thickness 50 cm	4.0 5.0	5.0 6.0	5.5 6.5
Paving of quarry stone, dry, laid in gravel bed 10 cm thick	thickness 20 cm thickness 25 cm thickness 30 cm thickness 40 cm	2.5 3.0 3.25 3.5	3.0 3.5 4.0 4.5	3.25 4.0 4.5 5.0
The same, using selected coarsely worked stones	thickness 20 cm thickness 25 cm thickness 30 cm thickness 40 cm	3.0 3.25 4.0 4.5	3.5 4.0 5.0 5.5	4.0 4.5 5.5 6.0

Table 65, continued

Type of lining			admissible v, ($m \cdot s^{-1}$) at medium depth (m)		
			0.4	1.0	2.0
Paving of quarry stone, dry, jointed with cement mortar and laid in gravel-sand bed, 10 cm thick	thickness	20 cm	3.25	4.0	4.5
		25 cm	3.5	4.5	5.0
		30 cm	4.5	5.5	6.0
		40 cm	5.0	6.0	6.5
Paving, using cement mortar 3 cm, jointed with cement mortar	thickness	20 cm	3.5	4.5	5.0
		25 cm	4.0	5.0	5.5
		30 cm	5.0	6.0	6.5
		40 cm	5.5	6.5	7.5
Paving laid onto concrete bed whose thickness is at least half that of the pavement; jointed with cement mortar	thickness	20 cm	4.5	5.5	6.0
		25 cm	5.0	6.0	6.5
		30 cm	5.5	6.5	7.5
		40 cm	6.0	7.0	8.0
Paving of concrete stones (solid prefabricates) laid dry in gravel-sand bed, 10 cm thick	thickness	10 cm	2.5	3.0	3.25
		15 cm	3.25	4.0	4.5
		20 cm	3.5	4.5	5.0
Concreting on slope with expansion joints filled with cement mortar, large reinforced concrete prefabs	thickness	10 cm	3.5	4.0	4.5
		15 cm	4.0	5.0	5.5
		20 cm	4.5	5.5	6.0
Cavity blocks of concrete			2.5	3.0	3.5
Retaining walls of concrete or reinforced concrete, concrete prefabricates		B 170	10.0	12.0	13.0

6.3.2.5.1 Bottom lining structures

Riprap. A mix of nongraded stones is used in this system of torrent-bed lining. The standard grain of this material is determined from the critical velocities or empiric formulas. The mix is heterogeneous, so the stones wedge each other and hold fast, forming a strong bottom resistant to the force of the stream. The layer is not smooth: the material is only spread over the smoothed surface of the bottom. To provide closer contact between the stones and to fill up the gaps between the larger ones, the thickness of the layer should be three times the size of the effective grain.

Stone pitching. This material has been overlooked in recent years though it offers a number of advantages: it is resistant to high flow velocities, its

surface is rough and its appearance is natural. Each stone is placed vertically and stone chips are used to fill the gaps and wedge the stones. Stone pitching is a good solution for narrow channels of a high bottom slope.

Stone pavement. Large quarry stones or stones picked up in or near the channel bed are the materials used in this method.

Pavement of quarry stone

Quarry stone is used for torrent channel bed lining in easily accessible places where a stronger lining is needed to protect the bottom against the carrying force of the water. In torrents whose bottoms consist of small-grained homogeneous sediment, no special bedding is needed to lay the stones on. A gravel sand bed has to be prepared in all other cases, the thickness of the bedding layer being 0.1 – 0.5 m (0.1 m under pavements 0.2 – 0.3 m thick, 0.15 m for those 0.4 – 0.5 m thick).

Dry pavement. A type of pavement in which the gaps are not filled and have to be wedged with stone chips; the alternative solution is to cut the contact surfaces of the stones to leave 20 – 30 mm gaps between them. If a stronger pavement is required, the gaps are sealed with cement mortar or are filled with the same mortar and pointed. The thickness of this type of pavement ranges between 0.2 and 0.5 m.

Pavement of stones laid in cement mortar. The stones are laid onto a layer of cement mortar, which is also used to fill the gaps which are then pointed. This type of channel bed lining is very hard and is used in torrent segments of a high channel-bed slope.

Pavement of large quarry stones. A type of paving suitable for lining in torrent segments where the stream is very strong. The stones are laid in place either dry or onto a bed of cement mortar. Cement mortar is then also used to point the gaps.

Pavement of stone found in and near the channel

The upper courses of torrents are not easily accessible to trucks to carry the paving material. Stones found in and near the narrow channels in such torrent segments can be used with advantage for the pavement lining. The advantages of such pavements include low cost, rough surface and a very natural appearance, as distinct from quarry stone. The collected stones are either laid dry or may also be laid into a wooden grillage.

Dry pavement. The same as with quarry stone. The thickness of the layer is 0.2 – 0.3 m.

Pavement with wooden grillage. Unworked collected stones are laid into a grillage consisting of wooden piles 0.19 m in diameter and 1 – 1.3 m in length, supporting logs 1.19 m in diameter (Fig. 113). This type of lining (0.25 – 0.40 m thick) is used in torrent segments of a high slope or in fords.

Pavement of large boulders

Dry pavement of large boulders (both quarry stones and boulders collected at and around the site) is a special type of lining pavement: as distinct from other types of stone paving, large boulders make a very strong layer, hence, they are mainly used in high-gradient segments of controlled torrents. No specially skilled

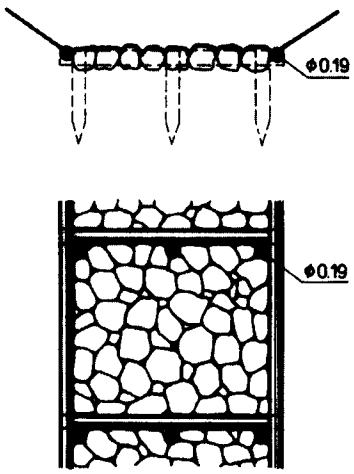


Fig.113 Dry paving laid in a wooden grillage.

labour is needed to build such a pavement: an excavator is used to prepare a bed for each boulder and to put the boulders in place. The boulders look natural, so this type of pavement can be included among those that meet the natural conditions of the channel.

6.3.2.5.2 Structures to line channel slopes

Channel slopes are lined to increase their resistance to the action of the water current forces at design discharge. Vegetation linings or combined linings are preferred in torrent control schemes. Other types of lining are only resorted to where vegetation lining or the combined approaches fail to provide sufficient resistance, e.g., in channels of steep slope, in sharp bends, etc.).

Vegetation lining

Vegetation lining and combined lining are among the environment-friendly interventions.

In addition to grassing, used to vegetate the side slopes, the most common approach is to use live osiers and brushwood of shrubby willows, exceptionally also other species.

Requirements for the quality of this live material depend primarily on the method by which the banks are to be lined and vegetated. First, it is necessary (regardless of the method) that the material be mature, live, healthy and fresh, free of any damage to the bark and buds. The main criteria by which quality is judged are the length of the osier and its diameter on the thicker end. Long nonbranching osiers at least 2 m long and 20–40 mm thick make the best material for wicker fences and willow brushwood covering. Shorter osiers may be used to vegetate lath fences, stone packings and ripraps. For fascine mattresses and for vegetated structures built to secure large bank scours, e.g., for fascines on piles, for brushwood grilles, for vegetated structures with riprap revement, etc., it is better to use longer and more amply branched osiers and brushwood which may have a greater

diameter. This gives the structure a higher mechanical strength and the greater roughness protects the banks by reducing the velocity, hence also the energy, of the stream.

The osiers to be used must grow fast and root and sprout well; hence, they have to be put in place at the right time.

During the year, every plant undergoes successive stages of development, each stage being characterised by a different rate of growth of the aerial and underground parts. The seasonal dynamics of the rooting and sprouting of propagated willows was studied by RASCHENDORFER (1953) who, on the basis of greenhouse and field experiments, arrived at the following conclusions:

Early spring (March) is a good season to propagate willows only in humid lowland areas. The growth of the plants propagated in spring is stronger in the aerial parts than in the rhizosphere. If drought occurs during this period, the root zone is (comparatively) too weak to support the large aerial part: the fresh sprouts soon exhaust the nutrient reserves from the "mother" osiers and die, though they may seem to have a good ecesis.

The willows propagated late in summer and early in autumn produce less roots (less absolute amount of root tissue), compared with early spring, but sprout growth is much weaker, or absent, in that period, so that roots can easily support the plant. Well rooted as they are, the willows are well prepared for the spring, the best season for the growth of sprouts. Spring is particularly suitable for willow propagation in areas where snow stays late into the spring.

Early summer (end of May; June, July) is unfavourable for willow propagation: willows produce the least amount of both aerial and root matter, hence, they show the poorest ecesis.

Winter is another season in which it is not recommended to plant willows. The material used for propagation remains dormant in the soil in winter and only resumes growth in spring when it takes root and sends out sprouts.

Willow material for vegetation lining of torrent banks should always only be harvested from healthy sources, using secateurs and saws. Overexploitation of the source trees or shrubs should be avoided, otherwise the plants' productivity would decline. From the tree willows the osiers should be cut close to the trunk, in shrubby willows the cut should be as close to the ground as possible. If osiers are harvested from willows growing within the stream channel, the cut should not be more than $0.05-0.10$ above the ground.

If the vegetative material is to be transported to longer distances, it is helpful (for easier handling) to bind them in faggots about 0.2 m in diameter. The freshly harvested willow material should be built into the vegetated structures as soon as possible. However, this is only possible with small structures: for large structures more material will often be harvested than can be put in place during a given shift. In such cases, the material must be properly stored to prevent it from drying out. The best places in which live willow brushwood can be stored in piles are depressions in the ground in shady places protected from the drying wind. If no such place is available near the site, the material can be covered with straw or with conifer brushwoods to protect it from sun and wind. The faggots may also be put in water in the channel: if the entire osiers are immersed in cold water, they remain fresh a long time without sprouting.

The willow brushwood cut in autumn can be stored longer without damage, as compared with those cut in spring. If they are cut in autumn to be used in spring, they can be stored in piles at the construction site and covered with snow. It is also recommended that winter-cut material be immediately brought to the site and covered with snow.

Osiers cut in summer need much more care than those cut in other seasons. Leaves must be removed and the osiers must be immediately put in place and irrigated. It is important in summer that the amount of osiers cut should not be larger than can be processed during a single shift. In spite of all care that may be given to the material in summer, the ecesis will never be more than about 30%. There will be large gaps between the willows in the stand and the plants will be delayed in growth, so fresh material will have to be added to fill the gaps – a costly procedure indeed. Hence, summer planting of willows for vegetation lining should be avoided and can only be resorted to in very exceptional cases.

If there are not enough live osiers from willows available, other woody species may be used instead to a limited extent (25%). Such replacement species, readily rooting and producing shoots from their adventitious buds, include poplar, hazel and ash.

Main willow species suitable for vegetation lining of torrent banks

The Willow is a wide genus comprising numerous species and hybrids; not all of them are suitable for vegetation lining of banks. The suitable willows must readily take root in vegetative propagation and must make a low and adequately dense brush of elastic and sporadically branching sprouts. These requirements are met by cultivated willows bred from the wild species. However, the majority of these species and hybrids only grow well if they have a copious supply of available nutrients. They need good care of the soil quality, possibly supported by fertilisation: on poor soils the cultivated willows remain stunted and their growth is reduced by weeds, so they can hardly do what is expected of them, i.e., protecting the banks of water courses.

The following shrub willows are the best for vegetaton lining and combined lining of banks:

Salix purpurea L., growing as a shrub to a height of 5 m. It usually occurs on gravelly deposits in submontane and montane areas at altitudes of up to 1200 m. It can be distinguished by its buds which are often facing each other at the end of the twigs. The leaves are short-stemmed, widest in their upper third, entire-edged near the stem and saggitate-dentate near the tip. Young twigs are purple red.

Salix viminalis L., forming bushy brush 2 – 5 m tall. Its natural spread is limited. It grows wild on soils copiously supplied with available nutrients. The maximum altitude of its spread is 700 m above the sea level. The willow sends out long and thinly branching sprouts with very long leaves which have a short stalk and a narrow and almost entire-edged blade.

Salix rubra L., a commonly occurring hybrid between the two willows mentioned above. It is a tall shrub with sprouts reddish at the ends. Its leaves look much like those of *Salix viminalis* L. This willow species thrives on light-textured loamy sands.

Salix triandra, a fast growing shrub, reaching a height of up to 5 m. Its branches are long and branching. Older plants have a grey bark which peels off in scales. Its leaves are long and have large stipules. It grows on damp sandy soils in submontane regions up to an altitude of about 700 m.

All these basic species, most frequently used for lining purposes, grow into tall shrubs which have to be lopped every 2−3 years, otherwise they would obstruct the flow of water in the channel. Maintaining such elastic stands is laborious and costly, so species producing shorter sprouts are being sought in which the cutting interval would be four years or longer. However, they must have an adequate form and type of branching and must root well and protect the bank reliably. Among the wide variety of willow species studied by the Czech Forestry Research Institute (MOTTL et al., 1979), the following have been identified as meeting these requirements:

Salix cordata, a short shrub about 1 m tall, with lodging branches. It readily takes root when used to protect torrent banks. Its root system is widely branched. *Salix cordata* is spread from lowlands to submontane areas.

Salix nicholsoni DIECK f. *purpuraescens*, a willow forming short and dense shrubs with widely branching osiers. It thrives on sandy loams and on gravelly soils with a medium to poor nutrient content. It can successfully compete with weeds.

Salix purpurea L. f. *gracilis* (*Salix purpurea "uralensis"*), a shrub growing to a height of about 2 m in several years. The brush formed by this species consists of thin, short and nonbranched or just slightly branched osiers. It is fastidious about the nutrients in the soil though it may even grow on gravelly soils in hilly areas and at high altitudes. Its growth is slow, so it can be cut at long intervals.

Good experimental results have also been obtained using *Salix humilis* MARCH., *Salix interior* ROW. and *Salix americana* HORT.

Influence of vegetation lining on flow rate in the channel

The aerial parts of the willows stands along the stream banks can substantially influence the discharge in the channel: the banks are coarser, reducing the channel's velocity and the cross-sectional area of the channel is reduced. The key hydraulic characteristics are influenced most significantly in narrow channels because the vegetated slopes take up a substantial part of the narrow channel's wetted perimeter.

FELKEL (1960) studied the effect of willow brushwood on water flow and found that in narrow channels overgrown with shrub willows, compared with channels free of willow brush, the flow rate is 18−70% lower. The main factors directly affecting flow rate include the willow-stand density and the cross-sectional velocity. Thinner growths contributed by 9−33% to the reduction of discharge. At low velocities, i.e., with a lower bed slope, the reduction of the discharge is much greater than at higher velocities.

The reduction of the volume of water that can get through the channel above the lined banks is not the only factor that contributes to the reduction of the discharge: the reduction of velocity (reaching 35−60% near the banks and 4−6% in the middle of the channel) is another important cause. Fig. 114 suggests that in actual projects using vegetation lining in narrow channels it is advisable not to include the parts above the bank slopes in the flow area.

The effect of willow stands on flow rates in wide channels was studied by JAŘABÁČ and ZELENÝ (1971) who arrived at the following conclusions:

− natural willow brush of irregular spacing reduced the discharge above the banks by 56%, whereas near the bottom the discharge was increased by 9.4%, the maximum being recorded in the middle of the channel (discharge increased by

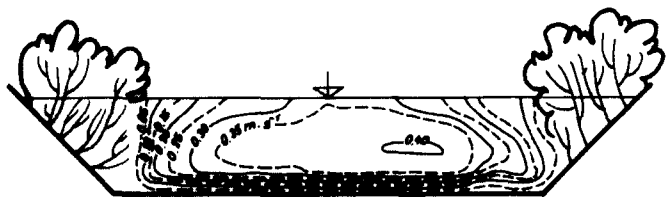


Fig.114 Effect of shrub willow stands on the distribution of velocities within the cross-section (after FELKEL, 1980).

23.7%). The overall reduction of the mean cross-sectional velocity caused by the willows lining the banks was 6.3%,

– willow brush raised the water level in the middle of the channel and, on the whole, reduced the flow area by 5.6%,

– the roughness coefficient of the slopes overgrown by natural willow brush was $n = 0.022-0.029$; in willow stands planted at a spacing of 0.2×0.2 m it ranged between $n = 0.036$ and $n = 0.038$ and at a spacing of 0.1×0.1 m it ranged between $n = 0.038$ and $n = 0.040$.

REIFERT (1977) studied the roughness of bank slopes (expressed in degrees of roughness [n]) and obtained $n = 0.059$ at a velocity of $1.2\ m \cdot s^{-1}$ and $n = 0.043$ at a velocity of $2.2\ m \cdot s^{-1}$. This confirms FELKEL'S conclusions that the effect of willow stands is greater at lower velocities.

Types of the vegetation lining of slopes

Grassing. There are two methods of grassing: seeding and sodding.

Seeding. In torrent control *grassing* is only used to protect the banks temporarily (e.g., in the upper part of banks to be lined with trees) and to cover the old filled channels or areas behind the bankline, if such areas are not designed for farming or forestry purposes.

Poor soils, sterile sands or gravels must have been covered with a $0.1-0.15$ m layer of humic soil before sowing.

The best species to be used for bank grassing are the deep and wide rooting perennials, including Dutch clover, smooth-stalked meadow grass, creeping fescue, meadow fescue, timothy, perennial ryegrass, meadow foxtail, Italian ryegrass, false oat and red clover.

Sodding. In torrent damming, sodding is primarily used to secure the strip (0.3 m wide at the minimum) between the strong lining at the toe of the slope and the upper part, grassed by sowing. (The whole slope is seldom lined by sodding). The sod is transplanted in blocks about 0.3×0.3 m in size. The thickness of the block must be sufficient to secure cohesion: a minimum thickness of 0.08 should be maintained if the block is taken from clay soil, 0.10 m from sandy soil and 0.12 m from cohesionless gravelly soil. Pegs are used to fix the sod on the slope.

The sod blocks are cut from grassy areas near the site where they are to be laid. Sod "peeling", handling and transport are difficult and laborious, so the blocks are now increasingly replaced by grass mats. Grass mats are sods precultivated in a $20-30$ cm layer of soil, peat, fluvial deposits etc., spread on a warp of any material allowing the roots to grow down through it, e.g., uvutan netting, jute and others.

The grass mats are precultivated on solid ground, e.g., concrete, plastic film, etc. They are transported in rolls which are then uncoiled either down the slope or along the direction of the stream.

Planting of willow cuttings. Willow cuttings may be planted over the whole width of the slope (Fig. 115), but this, however, is only done on less endangered banks in the straight segments of water courses and, especially, in the convex parts of curves. In concave curves or on banks exposed to a greater stress and in all other parts of the channel, the willow cuttings are only planted above the toe of a bank lined by a stronger lining.

Willow cuttings can also be used to vegetate ripraps or to replace withered parts of other types of vegetation (or combined) lining (e.g., wicker fences, willow brushwood covering, vegetated lath fences, etc.).

Cuttings are parts of osiers 10 − 30 mm in diameter and 0.3 − 0.5 m in length. Shorter cuttings are planted in the lower part of the slope, longer cuttings are planted in the upper parts, especially in gravelly soils. The spacing of the rows should be 0.3 − 0.5 m and within the rows the cuttings should be 0.3 m apart. Holes inclined upstream should have been made, using a steel dibble, before planting and the thicker ends of the cuttings should be put into the holes. In dry soils only the tip (one bud) should be left sticking out of the ground, in other soils the part above the ground should be 0.05 − 0.1 m long.

Fascine bundles. Bundles of live osiers 0.08 − 0.12 m in diameter, tied with burnt wire every 0.3 m. The osiers should be 20 − 30 mm in diameter at the thicker end and should be as long as possible. Brush bundle is most frequently used as a structural feature in willow brushwood covering.

Fascine roll. Also a bundle of live osiers. It is 0.20 − 0.30 m thick and may be as long as necessary. It is used either alone to line the toe of bank slopes or as a structural feature in fascine-gravel structures.

Fascine Pole. Another type of osier bundle, having a diameter of 0.20 − 0.30 m. It may have any length that may be required. The osiers may be thicker than in the fascine roll (preferably 20 − 40 mm on the thicker end). Thick, hard and nonelastic osiers are put in the middle of the bundle and the thin and flexible ones form the outer surface. Brushwood of the other woody species may

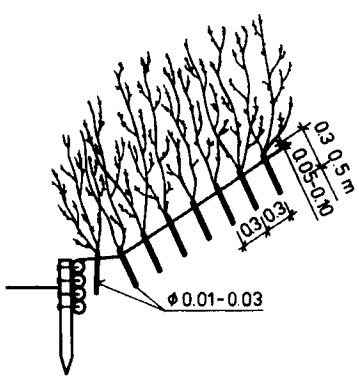


Fig.115 Planting willow cuttings on channel slopes.

be used to make fascine cylinders not required to vegetate. However, such poles may only be placed in the foundations of vegetated structures and in the inner parts of fascine-gravel structures inaccessible to air, which would cause rapid decay.

The procedure of making fascine poles of smaller diameters is described by PRÜCKNER (1965): in a flat place, drive stakes into the ground in two parallel lines 1.0 – 1.2 m apart. Then lay straps about 1.5 m apart across the rows of stakes and lay long and branched osiers on the straps. Osiers that are not straight should be placed with their tips pointing inside the roll. If they are not treated in this way, both the thick ends and tips of such osiers would stick out of the pole and obstruct handling. Separate long osiers are put onto one another so as to form the jacket of the hollow pole. Shorter branches may be put inside the cylinder but the cylinder should remain hollow. The thin ends of the shorter osiers should overlap with the thick ends of others. The thin ends and side sprouts of amply branching osiers inside the cylinder should fill the space among the thick ends of the long osiers of the jacket, otherwise the product would be a truncated cone, not a cylinder.

When the required amount of brushwood is put in place, attach other long osiers to the surface to make the outer surface and to give the pole its final shape. Then pull the material together: first just slightly, to make it possible to adjust some incorrectly placed osiers and then squeeze it tight to give the pole an actually circular shape and sufficient strength.

Finally, lay binding wire of adequate thickness next to each strap and tie the pole with it; the straps can now be removed.

If fascine poles are to be vegetated, though the bank onto which they are to be laid is covered by sterile gravel, it is recommended to spread several shovelfulls of wet soil onto the brushwood that fills the inside of the cylinder to provide better conditions for the osiers to root and sprout and to prevent them from drying out. Fascine poles are used either as separate lining elements to reinforce the toe of endangered banks (Fig. 116) and to eliminate bank scours on nonregulated streams, or as separate structural elements in fascine-gravel structures.

Fascine-gravel pole. Fascine-gravel poles consist of an outer layer of osiers and brushwood which encloses a gravel or stone filling inside. The diameter of such cylinders is 0.8 – 1.0 m, including the outer layer which should be 0.15 – 0.20 m thick. The use of osiers and brushwood should follow the same guidelines as in making "empty" fascine poles. The hollow inside the pole is usually filled with river gravel containing not more than 20% of sand. Large cobbles or refuse quarry stone may also be used as filling material. Quarry stone is preferred if the pole is to be heavy. The pole can be as long as required and have coniform endings so as not to allow the filling to fall out. Every 0.5 m they are tied together with a double ring of burnt wire 3 – 4 mm thick.

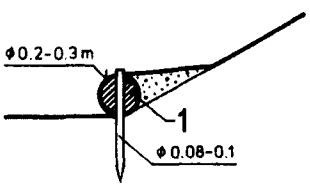


Fig.116 Fascine pole: 1 – fresh (live) osiers.

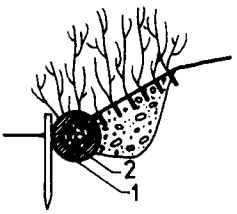


Fig.117 Fascine-gravel pole:
1 – live osier coating,
2 – gravel or stone filling.

Fascine-gravel poles are mainly used for local revetment of the banks, to secure the toe of slopes along torrents having a higher permanent water level, to stabilise bank scours (Fig. 117), or as structural elements in fascine structures. Though they are heavy they are flexible, capable of adjustment to uneven surfaces and to changes. As such, they are particularly suitable for torrents with nonstable bottoms (to be placed in concave parts of curves). If they are to vegetate, they cannot be installed in places where they would lie bare and exposed to abrasion by the material carried by the water (riprap can be used in such places to protect the pole). Fascine-gravel poles are laid in a trench and are protected by piles on both sides; the piles are 0.16–0.18 m in diameter and are driven into the ground 1.5 m apart.

Wicker fence. Wicker fences consist of stakes between which willow brushwood is wattled. The stakes, 0.08–0.1 m in diameter and 0.5–0.6 m apart, are driven into a trench dug in advance to a depth required for sufficient anchorage. Willow is the best material for the stakes but other locally available materials may also be used. The fence's height above the bottom is 0.3 m at the maximum and the depth under the bottom level is also 0.3 m. In concave parts of curves, the depth of the fence under the bottom can be appropriately increased. Fresh (live) willow brushwood 20–50 mm in diameter at the thicker end are wattled into the fence. The underground (i.e., under-the-bottom) part of the fence may be made of brushwood of various woody species, but above the bottom the material must be willow. Only in exceptional cases, if sufficient willow material is not available, can other sources be used but they should not form more than 25% of all material used. The fence will hold backfill which is levelled either horizontally or to a 1:8 gradient.

Wicker fences may be single or double (Fig. 118). Single fences or the first line of the double fence are located at the toe of the slope so that the connecting line of the fence's crests may run parallel to the slope line. Wicker fences are used on torrents where the bottom consists of coarse gravel and where there is no signifi-

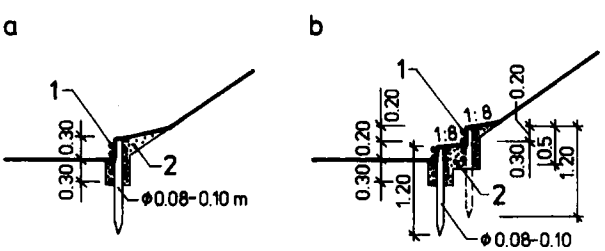


Fig.118 Wicker fence: a – one-row, b – two-row, 1 – the wattle, 2 – the filling.

cant danger of abrasion by the material carried by the water. The part of the bank above the fence should be lined by another method of vegetation lining, e.g., by planting willow cuttings. Wicker fencing is seldom used as a separate lining system because it may lead to the undesirable formation of whirlpools.

Willow brushwood covering. Willow bushwood covering is a longitudinal system of torrent lining, used mainly in the regulation of gravel-bearing water courses if nonstable gravels on the banks have to be stabilised as rapidly as possible. The osiers selected for the covering must be ripe, long and straight. Willow brushwood covering is used on banks adjusted to a gradient of 1 : 1.5 at the minimum, preferably 1 : 2, 1 : 2.5 to 1 : 3. The osiers are laid either tightly close to each other or at a minimum spacing (not more than 30 mm).

The best length of the osiers is 2 – 3 m. With narrow covers the overlapping ends are cut back. If the covering is to be wider, the thicker ends of the osiers of each subsequent strip should overlap the strip below by about 0.3 m.

If sufficient sources of willow material are not available, fresh (live) brushwood of other readily rooting and sprouting woody species may be used, but not more than 25% (e.g., hazel, alder, poplar and ash).

On water courses whose water level declines significantly during the growing season and in which the whole channel often dries up completely (e.g., torrents and gravel-bearing streams in flysch areas) the thicker ends of the osiers (not more than 40 mm in diameter) should be buried to a depth of 0.3 to 0.5 m in a longitudinal trench in the bottom where they are protected from drying. Their ends should never project into the flow profile: parts of osiers constantly and directly exposed to water produce roots and sprouts much more rapidly than the parts lying on the drier parts of the slope which may even fail to take root (BAUCH and LINKE, 1964).

The thickness of the willow brushwood covering depends on the conditions under which the cover is built. In regulated torrent segments or on water courses where the covering is exposed to less stress, one layer of osiers will be sufficient but where the stream is more vigorous and produces great mechanical stress, the cover will have to be 0.05 to 0.1 m thick.

The covering has to be anchored at the toe of the slope, depending on the location and on the stress to which it is exposed. Burnt wire fastened to stakes or a fascine bundle will suffice in convex parts of curves on which no stress is exerted. In the straight segments of the stream and especially on the concave sides of curves where the stress is high, wicker or lath fences (Fig. 119) have to be built, the thicker

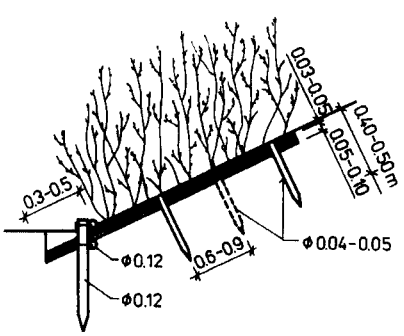


Fig.119 Willow brushwood covering.

ends of the covering osiers being wattled into the wicker fence or fixed between two laths in the lath fence. Double or treble laths can be used in the latter type of fence if necessary, depending on the stress to which the bank being protected is exposed.

For fixing the osiers to the slope, there are several options to choose from. The simplest way is to use wire 2 – 3 mm in diameter, tied to stakes 40 – 50 mm thick and 0.4 – 0.5 m long. The stakes are driven into the ground in rows at a 0.6 – 0.9 m spacing, forming a chessboard pattern at distances of 0.1 – 1.5 m.

In the Beskid Mountains on the Czech-Polish border, osier covering is fixed to the ground by means of fascine bundles which are laid on the slope at a spacing of 0.6 – 0.9 m and fastened with wooden stakes 60 – 80 mm thick and 0.6 – 0.9 m long. Another Beskid option is to use two osiers wattled between a line of stakes 50 mm thick and up to 1.0 m long, driven into the ground 0.8 m apart; the line of stakes runs obliquely upstream over the slope.

To protect the willow osier covering against drying out and to support rooting, a 30 – 70 mm layer of earth is spread over the finished and fixed covering. If necessary (on gravel or on poor soils), another layer of earth may have been spread under the osiers.

Willow osier mats. Willow osier mats eliminate some of the shortcomings of osier covering: it reduces laboriousness and simplifies handling. Much less time and less material is required to build this type of bank lining. The mats, 1.5 – 1.8 m wide and 10 – 15 m long, are woven by a machine, using annual willow osiers and brushwood.

The mats are made in winter and can remain alive until May if kept under suitable storage conditions. Stakes are used to fasten them onto the slope (in the same way as with willow brushwood covering).

Nonvegetation lining

Lath fence. Lath fences consist of laths nailed to stakes which are driven into a trench about 0.3 m wide; the depth depends on how deep the lath wall is to reach under the level of the bottom (Fig. 120). The stakes are usually made from softwood (preferably spruce) and are 0.1 – 0.12 m in diameter; the stakes' length depends on the required depth of anchoring and they should be 1.5 m apart. Softwood laths (diameter 0.12 m at the thick end and at least 60 mm at the other end) are nailed to the stakes. Gravel is then used as filling behind the fence. The maximum height of lath fences is 0.3 m above the bottom. The depth to which the fence reaches depends on its location (it should reach deep under the bottom on

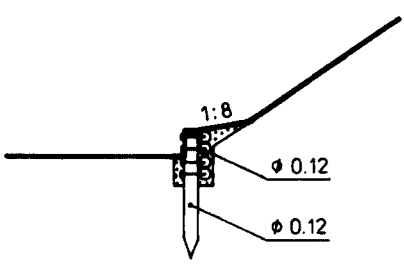


Fig.120 Lath fence.

the concave side of curves and less deep on the convex side and in straight segments of the channel), on the carrying force of the stream and on the material of which the bottom consists.

Lath fences are erected to protect the toe of slopes lined by vegetation or by another method of lining on streams of a higher slope, with coarser bottom load (where heavier abrasion can be expected) and in places exposed to the greatest water pressure. Lath fences are also often built during maintenance work.

On torrents of a high gradient and high flow velocity where stronger lining is required, it is useful to protect the toe of the slope by means of a more massive structure of logs. The log wall should remain under the level of base flow and should support dry stone pavement or stone packing (of large pieces of stone).

Stone pavement. Different types of stone pavement (the same as with stream-bed lining) are used for the lining of bank slopes (Fig 121). The paving is supported by a toe of stonework masonry with a foundation deep enough below the level of the bottom. If the bottom itself is also lined by stone pavement, no masonry toe is required and the paving of the bottom passes into the paving on the slopes. However, if a road runs parallel with the stream, it is useful to build a masonry toe to divide the paving of the bottom from that of the slope.

If large boulders are used as paving material, the pavement will be very strong and the torrent will have a very "natural" appearance.

Riprap. Riprap serves for the lining of the toe of the slope in places exposed to a high pressure of the water flow, especially in the concave parts of curves. It is used with advantage in maintenance work, e.g., to secure the toe of the slope when bank scours are to be repaired. The quarry stone used as material is spread so as to attain a gradient of 1:1.25 to 1:2 (Fig. 122).

Stone packing. In torrent control stone packing (Fig. 123) serves to line the toe of slopes which themselves are lined by a lighter type of lining; it may also be used to line the whole slope, e.g., in concave parts of bends. It is made of unworked quarry stone or stones found near the site. Gravel and sand are used to fill the gaps between the stones. Where the stones are in contact with each other, the gaps are wedged with stone chips.

To be well anchored, stone packing should reach to a sufficient depth under the level of the bottom. The incline of the surface should be 1:1 at the minimum. Stone packing is a good solution for continuous lining systems in which it secures the upper part of a slope, lined by trees. Good results are also obtained where packing (using large stones) serves for the lining of the whole slope to secure shorter bank scours (up to a gradient of 2:1).

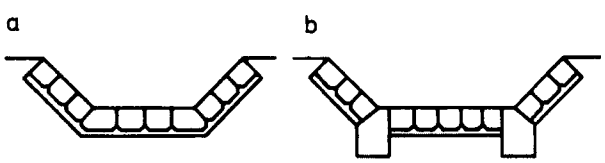


Fig.121 Stone pavement: a – pavement on the slopes and bottom, b – pavement on the slopes supported by slope toe lined with stone masonry.

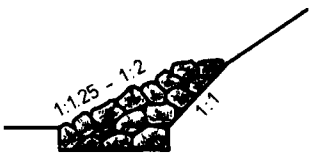


Fig. 122 Riprap.

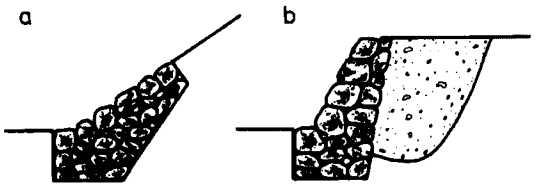


Fig.123 Stone packing: a — in slope toe, b — securing small bank scour.

Narrow channels of a high slope can be lined with stone packing within the whole cross profile. However, large boulders must be used in such cases to make a very strong and rough lining.

At the toe of the slope, mechanised technology can be used to build stone-packing reinforcement (similar to the boulder pavement technology). Natural vegetation soon spreads over the slope above the water level, giving the stream segments lined by stone packing a natural appearance.

Stone-filled gabion lining. The main purpose for which stone-filled gabion elements (Fig. 124) are used is to secure the toe of slopes or to build retaining walls of roads running parallel to the water stream. They are also used with advantage to line the whole slope of a channel. Such a lining is flexible and capable of adapting to changes that may occur in the channel bed.

The main element in the stone-filled gabion structures is a prism-shaped cage with a rectangular or square base. The gabions are made from wire netting of square mesh. Point-welded zinc-coated nets are manufactured for these purposes in some countries: their shapes are adjusted to their use for assembling gravel-gabion elements or structural elements to build gravel-gabion structures. To make a gabion, the cage walls are fixed together using a wire spiral or a wire drawn through the meshes on the edges of each part. Point-welded nets which are not zinc-coated, such as those manufactured by Czech producers, are prone to corrosion but they are thick enough to provide a sufficiently long service life.

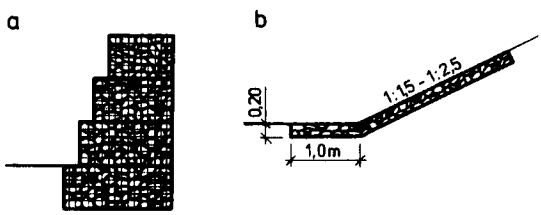


Fig.124 Stone-filled gabion lining: a — retaining wall, b — mattress.

Gabions 3−4 m long are tied together with wire at the ends and are filled with gravel or stone. The stone filling must be carefully distributed inside the cage, especially in the corners and near the walls. Wires placed horizontally in the gabion provide additional strength.

A gabion placed in the foundation of a gravel-gabion structure is called a mattress. Also referred to as mattresses are gravel gabions used to line channel-bed slopes. Mattresses are laid on the levelled slope across the axis of the stream and are supported by toe revetment (gravel gabions, walls of stonework and cement mortar or thick stone packing). The width of the mattresses is 2.0 m and their thickness should be at least 0.2 m. Once they are filled with gravel they are closed with a wire netting cap. The retaining walls of gravel gabions are either rectangular on cross section or are vertical on the back side and sloping back on the front side.

Log walls. Log walls are built to eliminate bank scours and to support parallel roads or steep banks, especially on torrents that carry coarse bed load.

The log structure consists of longitudinal and cross logs laid onto bolsters and fixed with wrought-iron nails. The front wall of the log structure has an inclination of 5:1 and the back side is vertical (Fig. 125). The space between the two walls is usually filled with stone collected at the building site or with rubble if the local sources do not provide sufficient filling material. Selected wedge-shaped pieces of stone are put into the gaps between the logs. The surface of the structure's crest is levelled to form paving. In order not to allow water to undercut the structure and carry away the stone filling, a wooden floor of pole-stage tree stems, 0.1−0.2 m in diameter, is built underneath. In single-wall log structures, the front wall is built at an inclination of 3:1, the other wall being replaced by ties nailed to cross logs and anchored in riprap.

Retaining walls. In torrent control, retaining walls are most frequently used in built-up areas where there is not enough room for less steep slopes, in

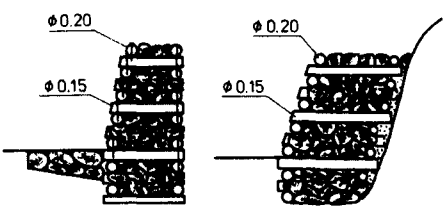


Fig.125 Log walls.

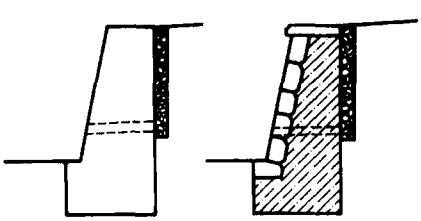


Fig.126 Retaining walls.

narrow channels where the inclination of the natural slopes is steeper than 1:1 and in places where the bank lining passes into the flank walls of bridges. In the upper segments of torrents not easily accessible to trucks, retaining walls are built from worked stones collected near the construction site.

Small wedge-shaped stones or chips are used to fill the gaps between the stones: near the front side of the wall, the wedges should be laid with their wider part inside the wall. The ground plane of the wall is usually trapezoidal, the front side inclining by 3:1 and the back side being vertical or inclined.

Retaining walls of quarry stone are usually built as random masonry, using cement mortar; in built-up areas cyclopean masonry is used. Such walls are trapezoidal in cross-section and the inclination of the side facing the water is 5:1 (Fig. 126).

Retaining walls of concrete have the same cross-section as stonework walls. To protect the front side against abrasion by bottom load, the exposed concrete surface is lined with quarry-stone masonry, either random or cyclopean. The crest of the wall is covered with a parapet slab of concrete.

The IZT 18/10 and IZT 19/10 prefabricated reinforced concrete units, shown in Fig. 127, can be used to line the face of concrete walls. The prefabricated unit has a trapezoidal cross-section and its face (incline 5:1) is irregularly corrugated. The concrete from which it is made guarantees resistance to abrasion. The prefabricates are laid onto concrete foundations covered with cement mortar. The triangular grooves in the sides of the prefabricated units are filled with cement mortar to fix the units together. When a line of prefabricated units is in place, a form is built on the rear side and the empty space is filled with concrete. Then another layer of prefabricated units is laid; its vertical joints should alternate with the joints of the layer below.

Drainage should be provided on the rear side of the retaining walls. Gravel filters protect the drain holes from clogging.

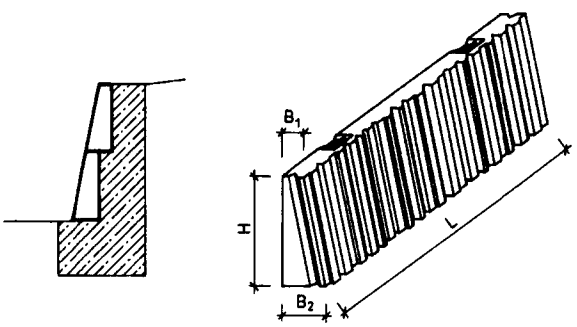


Fig.127 Concrete wall with IZT prefab units lining the face.

Combined lining

Vegetated riprap. Vegetated riprap is built in the same way as riprap without vegetation. Two methods can be used to vegetate the stone-covered surface; both have been well tested in torrent control systems (Fig. 128):

– gravel is used to fill the gaps between the stones and thick willow cuttings 30–50 mm in diameter and 0.5 m in length are stuck into it.

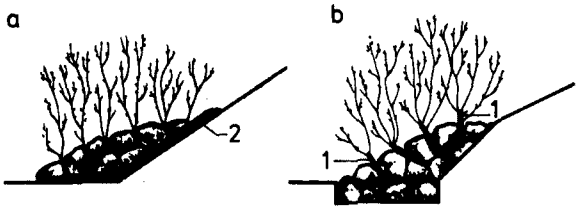


Fig.128 Vegetated riprap: a – planted on a bedding of brushwood, b – with willow cuttings, 1 – willow cuttings, 2 – bedding of willow brushwood.

– litter or live willow brushwood is spread onto the slope, levelled to the required gradient, and the stones are put onto it. In this case, the willow brushwood is in better contact with the ground and with water, so it has better conditions for rooting. The sprouts find their way through the gaps between the stones.

Vegetated riprap is a good lining system for the toe of slopes lined by another vegetation lining method (e.g., planted with willow cuttings or lined by willow brushwood covering). Used mainly in concave bends.

Vegetated stone packing. Vegetated stone packing (Fig. 129) is built in the same way as bare stone packing. Large stones are placed onto the surface, the inner spaces being filled with smaller stones. The surface stones are laid onto a bedding and the gaps between the stones are filled with earth and live willow brushwood, laid with the thicker ends pointing towards the stream.

Vegetated stone packing is used to protect concave banks or to secure shorter and shallow bank scours. With its flexibility it is particularly suitable in locations where the lining can be expected to sink as the bottom deepens. In the upper course segments of torrents not easily accessible to trucks the material has to be provided by collecting from local sources; quarry stone can be used in the segments lower downstream and in accessible places where local sources are not sufficient.

Vegetated walls. Retaining walls consisting of a cage of logs or prefabricated units filled with stone can be vegetated by placing fresh branches in the structure (Fig. 130). The branches, taken from woody species of a high sprouting capacity (e.g., ash, alder, willow and others) are laid onto the stone filling during construction. On the back side of the wall, the thicker ends of the branches are stuck into the ground and on the front side a third of their length is left to project into the flow profile. For better rooting, the gaps between the stones have to be filled with earth.

Vegetated stone-filled gabion mattresses. A gravel gabion mattress (Fig. 131) can be vegetated by grassing or by establishing a stand of shrub-type

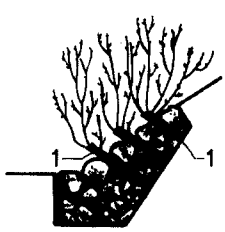


Fig.129 Vegetated stone packing: 1 – live osiers.

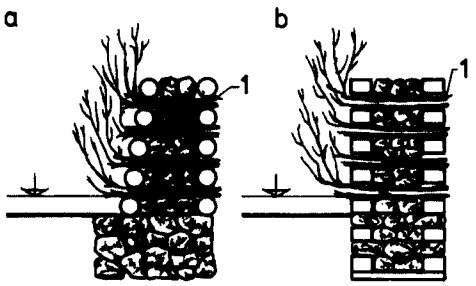


Fig.130 Vegetated walls: a — vegetated log wall, b — vegetated wall of prefabricated units, 1 — live branches of species of a high sprouting capacity.

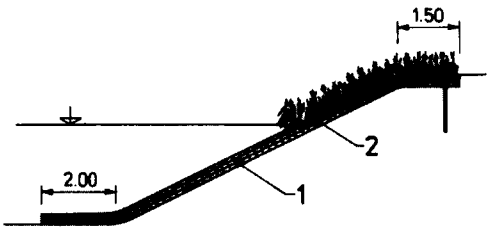


Fig.131 Vegetated stone-filled gabion mattress: 1 — stone filled gabion mattress, 2 — willow pad.

willows. For grassing, the grass seeds are sown onto a 0.05—0.10 m layer of soil spread over the mattress. For a shrub stand, willow cuttings are put into the gravel filling with their thicker end reaching into the ground. The mattress can also be vegetated using a willow pad laid on a smoothed surface of the slope under the mattress.

"Semi-vegetation" blocks. Concrete blocks with holes to be sown with grass or planted with willow cuttings are used mainly for the reinforcement of the toe of the slope in the straight segments of regulated torrents. They are supported by a stone wall or a fence of thick poles or logs (Fig. 132).

Vegetated lath fences. Vegetated sets of lath fences (Fig. 133) are several lines of lath fences with willow brushwood laid between them. The fences

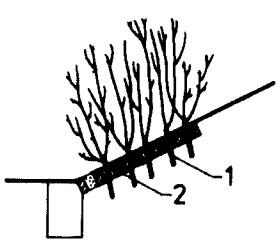


Fig.132 "Semivegetation blocks"
1 — willow cuttings, 2 — earth filling.

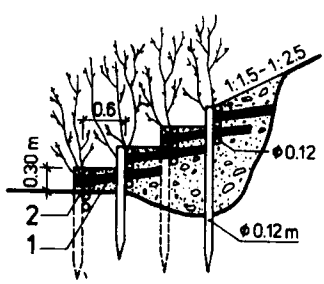


Fig.133 Vegetated lath fences:
1 — live willow brushwood, 2 — earth filling.

are built one above another so that the connecting line of their crests has an inclination of 1 : 1.5 or less. Fresh willow brushwood forming a layer 0.05 – 0.10 m thick is drawn between the upper two laths of each fence. The brushwood is laid with its thicker end pointing to the stream and are covered with a 0.05 – 0.10 m layer of soil.

Vegetated sets of lath fences are primarily used to secure scours in banks. They may also be built in places requiring a stronger lining within compact vegetation lining systems in longer tracts of banks.

6.3.2.6 Structures for torrent control

Structures built for the control of torrents are designed to secure the stability of both lined and unlined segments of the channel bed, to get over the differences in channel-bed altitudes in slope-stabilisation projects, to catch the bottom load in the sedimentation zones and to realign the water stream.

Transverse structures for torrent control are structures built across the torrent channel to stabilise the level of the stream bed, to adjust the bed slope and to catch sediment.

Structures stabilising the level of the stream bed – stabilisation belts

Stabilisation belts are structures whose crest lies at the level of the torrent bottom. Their purpose is to prevent excessive deepening of the bed in unlined channels or to secure the stability of the lining elements in lined channels.

Between the stabilisation belts of an unlined channel bed, the loose sediment particles will be removed in a natural process, leading to the formation of a stable channel-bed slope. The material forming the channel bed is caught between the stabilisation belts which cut the bed into shorter segments, thus reducing bottom deepening to an admissible level (Fig. 134). As the channel-bed slope stabilises, the stabilising bands become spillway structures under which scours develop. The scours should not reach deeper than 0.4 m above the foundation of the stabilisation belt, otherwise the band's stability would be threatened. Riprap can be used to fill scours to prevent deepening.

The distance between the stabilisation bands can be determined from the following equation:

$$l = \frac{\Delta t}{i - i_s}, \quad \text{(m)} \tag{6.111}$$

where Δt – maximum admissible level of stream channel deepening (m),
i – original channel-bed slope,
i_s – stable channel-bed slope.

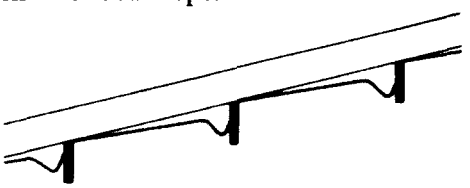


Fig.134 Division of the channel bed by stabilising bands.

The depth of the scour can be calculated from the adjusted *Eggenberger-Müller equation* (GRUND, 1970):

$$t = w \frac{\Delta H^{0.5} q^{0.6}}{d_{90}^{0.4}} - y_d, \qquad \text{(m)} \tag{6.112}$$

where w – coefficient $w = 0.4$,
ΔH – height difference between the energy line in the channel upstream from the stabilisation belt and the water level downstream from the belt (m),
q – the specific flow rate ($m^2 \cdot s^{-1}$),
d_{90} – grain size corresponding to 90 % of the value on the grain-size curve of the sediment downstream from the stabilisation belt,
y_d – depth of water in the channel downstream from the belt (m).

The distance of the place with the maximum scour depth from the spillway edge is calculated according to GRUND'S (1970) equation:

$$L_{max} = 7.5t \qquad \text{(m)}. \tag{6.113}$$

Stabilisation belts are especially suitable for the control of torrents with a bottom consisting of more or less monodispersive sediment: this applies to the control of whole stretches of torrents as well as to limited local interventions. The belts can also be built in natural torrents where bottom deepening is to be prevented without further regulation of the cross section and without realignment; sometimes they may also be a good solution for maintenance.

Example 4

The torrent channel bed has the following parameters: $b = 6.0$ m, $y = 1.0$ m, $m = 1$, $S = 7.0$ m, $R = 0.79$ m, $E_d = 1.39$ m, $i = 0.018$, $i_s = 0.0128$, $Q = 19.32\ m^3 \cdot s^{-1}$, $d_m = 0.10$ m, $d_{90} = 0.13$ m.

The admissible level to which the channel bed can be deepened: $\Delta t = 0.3$ m.

The distance between the stabilisation belts:

$$l = \frac{\Delta t}{i - i_s} = \frac{0.3}{0.018 - 0.0128} = 57.69 \text{ m}$$

Scour depth:

$$t = w \frac{\Delta H^{0.5} q^{0.6}}{d_{90}^{0.4}} - y_d$$

$$\Delta H = E_d + \Delta t - y_d = 1.39 + 0.3 - 1.0 = 0.69 \text{ m},$$

$$q = \frac{19.32}{7.0} = 2.76\ m^2 \cdot s^{-1}.$$

$$t = 0.4 \frac{0.69^{0.5} \cdot 2.76^{0.6}}{0.13^{0.4}} - 1.0 = 0.38 \text{ m}.$$

The minimum depth of the foundations of the stabilisation belts:

$$p = t + 0.4 = 0.38 + 0.4 = 0.78 \text{ m}.$$

The distance from the place with the maximum scour depth t:

$$L_{max} = 7.5t = 7.5 \cdot 0.38 = 2.85 \text{ m}.$$

The structural form of the stabilisation belts depends on the building material used. Stonework, concrete, gravel gabion or rock belts are used in systematic control measures covering long segments; wooden belts (sleepers) and loose rock belts are built to secure the bottoms of natural channels or to provide channel maintenance.

Stonework and concrete stabilisation belts

Stabilisation belts of this type form retaining and stabilising ribs anchored in the bottom and banks of the channel. The belts are keyed into the banks in a retreating manner and the crest of the flank of the belt normally reaches up to the height of bank lining. Both stonework and concrete belts are rectangular in cross-section. The stonework belts are made from stone masonry and cement mortar and are 0.5 m wide at the crest. Concrete belts are narrower at the crest (0.3 m) (Fig. 135). Where the concrete belt's surface is in contact with water, it is protected by a lining of quarry stone and cement mortar.

Stonework and concrete stabilisation belts are most frequently built as retaining and stabilising elements on bottoms lined with stone paving.

Stone-filled gabion belts

Stone-filled gabion belts are a flexible stabilisation feature used with advantage in torrents whose channel bed consists of small monodispersive sediment. The belts are built from wire cages rectangular in cross-section which are filled with coarse gravel or rock collected at and near the site. Their main advantage in comparison with stonework and concrete belts is the flexibility of their structure, allowing them to adapt safely to the changes that may occur with the formation of a scour. They are particularly useful as separate structural elements in local interventions and in maintenance work though they may also be designed for systematic training schemes.

Wooden belts

Wooden belts (Fig. 136) can be used for stabilisation purposes in both lined and nonlined channel beds. They are particularly suitable to secure the channel-bed lining of dry-stone paving in local interventions and maintenance work.

Simple wooden belts are made from logs up to 0.3 m in diameter. They are laid into a trench dug across the channel bed and are fixed in the banks to a depth of 1 m. In narrow channels, the logs are held against a buttress pile driven into the

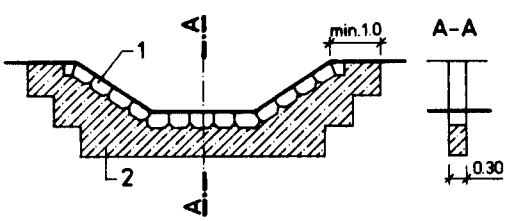


Fig.135 A concrete stabilising belt: 1 — lining of quarry-stone masonry, 2 — concrete.

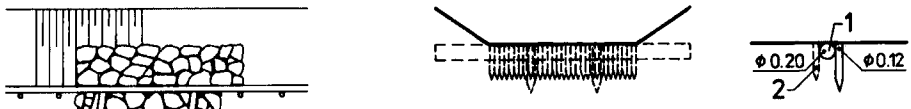


Fig.136 Wooden band:
1 – logs,
2 – wall of wooden stakes.

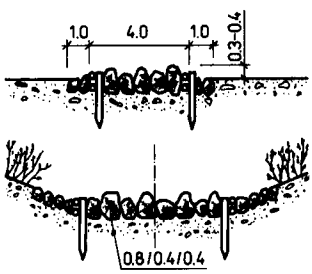


Fig.137 Rock belt.

bottom in the middle of the channel bed. In wider channels the piles are 1.5 m apart.

Double belts are built from double logs. They are used when the natural process of gradient development is expected to lead to a greater deepening of the bottom.

To prevent scours beneath the wooden belts from spreading upstream, thereby undercutting the belt, it is recommended that a wall of wooden stakes be installed on the upstream side; the stakes have to be driven to a sufficient depth under the expected level of the scour's bottom. Once the scour has developed, it is recommended to fill it with riprap.

Rock belts

Rock belts consist of stone packing (Fig. 137). The maximum size of the rocks is 0.7 – 0.9 m. The rocks are laid vertically with half their length sticking out above the level of the channel bed. On both ends the stone packing is fixed with wooden belts and riprap; on the banks it extends up to half the height of the lining. The rock belts' function is wider than that of other types of stabilisation belts: they increase the roughness of the wetted perimeter, thus reducing the kinetic energy of the water current. They have a natural appearance, so they are included among the environment-friendly interventions.

Structures for channel-bed slope control

The structures to control the slope of channel beds include sills, weirs and chutes.

Sills

Sills are low transverse structures whose height does not exceed 0.3 m and whose crest is flush with the upstream bottom. Low as they are, they need no stilling basin and can make do with an apron to protect the downstream floor. Stone and wood are the most common materials for building sills.

Stonework sills are, in essence, low weirs with an apron on the downstream side. They are most frequently built within torrent-control schemes with banks protected by retaining walls or stone paving (Fig. 138).

Wooden sills are the most widespread type of sill structure. They are most effective on torrents of a lower slope in which the sediment consists of small particles. The sill body is made from one or two round woods (diameter 0.19 − 0.20 m) which are fastened to piles (diameter 0.15 m). The apron is lined with material collected around the construction site: brushwood, wood or stone. A bedding of brushwood or round woods will suffice particularly on torrents where permanent water flow prevails, where the channel-bed slope is mild and where bottom load consist of small particles. The layer of brushwood should be 0.1 − 0.2 m thick and should be fixed under the sill body; on the other end it is fixed between two cross pole-stages 0.12 m in diameter (Fig. 139). Stakes have to be driven into the bottom and the layer of brushwood forming the apron should be tied to them with wire.

Fig.138 Adjustment of the channel bed slope by stonework sills (Photo by L. Novák).

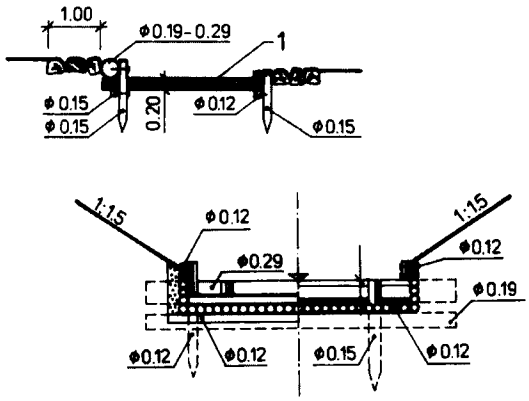


Fig.139 Wooden sill with a brushwood bed serving as apron (1).

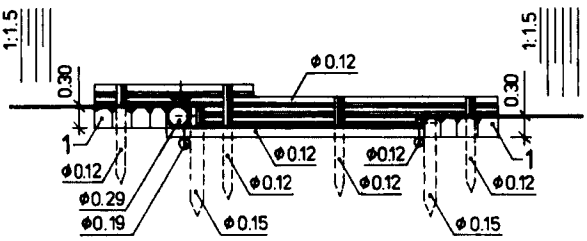


Fig.140 Wooden sill with an apron of logs: 1 — dry stone paving.

An apron of round woods (Fig. 140) is fixed to cross pole-stages nailed on to stakes. The cross pole-stage on the downstream end maintains a pool of water on the apron, thus preventing the wood from decay.

On torrents carrying a coarser bed load and having a higher channel slope, the water overflowing over the crest has a high kinetic energy, so the apron must be strong. The best apron surface is stone packing, each stone being put in place vertically (Fig. 141): such a floor lining is strong and rough and able to check the excess kinetic energy of the water current so that the apron need not be too long. A round wood fastened to stakes holds the stones in place, not allowing them to become loosened. To prevent scouring, it is useful to line the bottom on both the upstream and downstream sides of the wooden sill with a 1 m band of dry paving or levelled riprap.

The channel banks above the spillway and above the apron are exposed to high stress and have to be adequately lined. If the sills are built in a channel where the

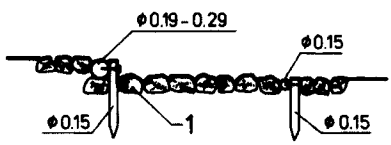


Fig.141 Wooden sill with an apron of stone packing: 1 — stone packing.

toe of the slope is lined by a lath fence, the fence should extend over the apron. There must be a strong lining, preferably stone packing, above the lath fence. Vegetated sets of lath fences have not proved to be a suitable type of bank protection above the sills because the water removes the filling from behind the fences. Stone packing is the best and strongest type of lining of the slopes on the downstream side of the sill.

Sills can also be built by a method corresponding to the natural development of the torrent channel bed. Austrian designers copied low sill structures from the natural formations developing in the channels of torrents: to build their sills they used combinations of quarry stones of different sizes. In narrow channels where bottom stability has to be secured by lining, it is recommended to form a stepped profile of the bottom as shown in Fig. 142. In smaller channels with unprotected bottoms it is useful to build a small sill about 0.25 m tall as shown in Fig. 143.

In wide torrential streams it is recommended to use large boulders to build the sill body, which passes into the apron on the downstream channel floor. On the upstream side the boulders rest against a wall of hewn wooden rods which prevent the leakage of the boulder structure (Fig. 144). Fig. 145 shows a low sill structure corresponding to the natural formation of a torrent channel in the Beskid Mountains. The sill consists of a flat or curved surface of stones collected in or near the channel. The stones are laid between logs fixed to piles. A loose rock apron should be installed below the sill on the downstream side.

Weirs

Weirs are transverse structures taller than 0.3 m (Fig. 146) whose crest is level with the upstream channel bottom. Most frequently they are parts of a larger torrent-control system but sometimes they may also be built as separate local consolidation structures. In the latter case, their crest need not be flush with the upstream channel bottom: the levelling may be attained later when the temporary catchment basin is filled with sediment. However, sediment accumulation is not the purpose for which they are built: their purpose is to modify the slope. In this they differ from check dams.

Fig.142 Natural "sills" in the channel bed.

Fig.143 Sill structure fitting into the naturally developed bottom.

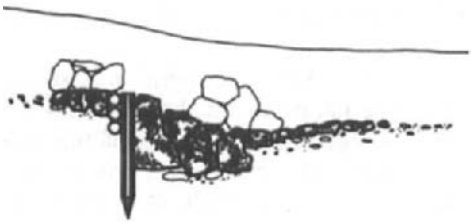


Fig.144 Sill structure of large boulders.

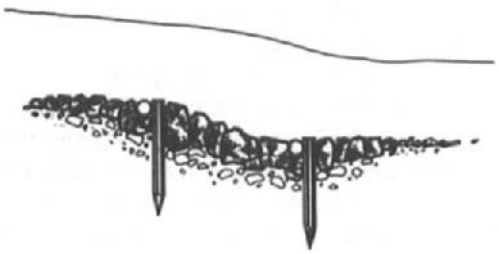


Fig.145 Sill structure: stones between logs fixed to stakes.

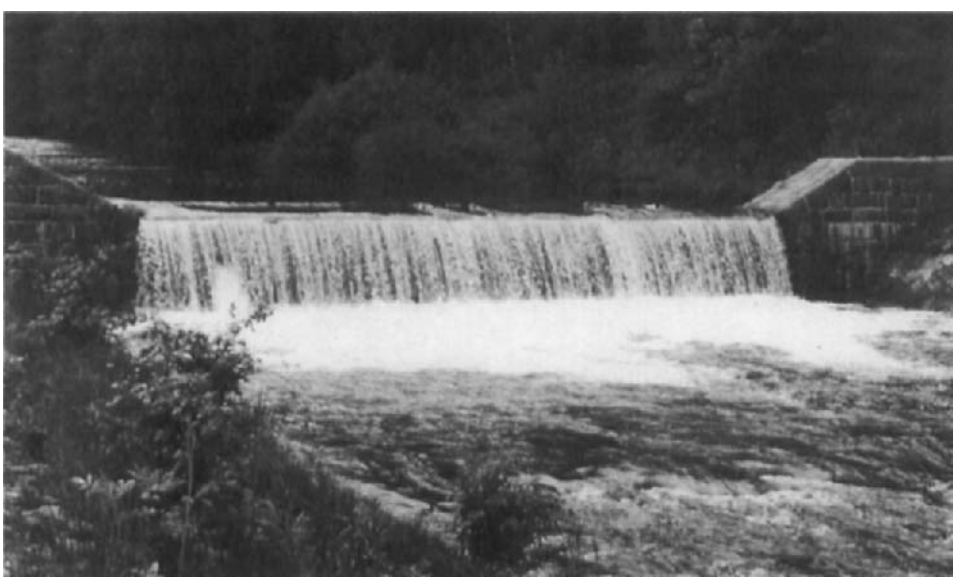


Fig.146 Stonework weir (Photo L. Novák).

The location of weirs within a torrent control system depends primarily on the requirements for modifying the channel-bed slope. Other factors that also have to be considered include the alignment of the regulated channel bed and the general

setting (especially in residential areas). Straight stretches of the channel bed offer the best locations for weirs.

If a weir is to be installed in a channel bend, its axis should be adjusted so as not to allow the overflowing nappe to hit the side slopes of the channel. On the concave side of the channel the lengthwise axis of the weir should be rotated by 17° (at the maximum) downstream and should not intersect the arc running at a distance of $b/3$ parallel to the convex bank (Fig. 147). A desirable trajectory of the nappe is reached by rotating the weir's axis so that it is perpendicular to the line connecting the midpoint of the spillway with the channel-bed centre line at the end sill (Fig. 148). The lengthwise axis of the end sill must have already been orientated to the centre of the bend.

The weir consists of the weir body and the apron.

The weir body. The weir body includes the middle part with the spillway and the sides (wing wall) which are founded into the banks. The depth of the foundation must be sufficient to prevent water from scouring and tunneling around the weir body. The depth of the foundation is greater under the bottom part of the structure and may be shallower near the crest, though it should never be less than 1.0 m under the surface.

The sides must be founded into natural earth; only in exceptional cases, if a very long and costly weir body would have to be built to meet this condition, may the wing wall end off in an embankment, e.g., in an original wide channel, etc. The

Fig.147 Location of a weir in a bend — theoretical principle.

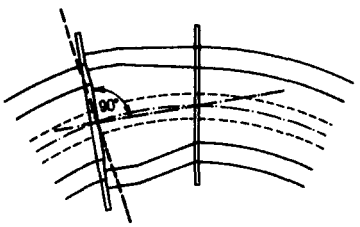


Fig.148 Location of a weir in a bend — practical realisation.

crest of the weir wing wall is horizontal. The shape of the spillway should correspond with the cross section of the channel upstream from the weir. The structure of the weir body depends on the material used: they may be made of masonry, concrete, prefabricated blocks or wood, or they can be combined, or made of stone-filled gabions.

Masonry Weirs. The body of the weir (Fig. 149) is built of stonework masonry. Rocks found at and around the site may be used to build dry rubble masonry in cases of local interventions in places inaccessible to trucks. In all other cases, the weirs should be built from quarry stone and cement mortar.

On the upstream side, the weir wall is vertical whereas the downstream side has a batter of 5:1; this may be so throughout the height of the weir but smaller weirs may have an inclination of 10:1 from the spillway edge in the wing walls.

Holes (0.1/0.2 to 0.2/0.3 m) are left in the body of stonework masonry weirs to drain the back side of the weir wall.

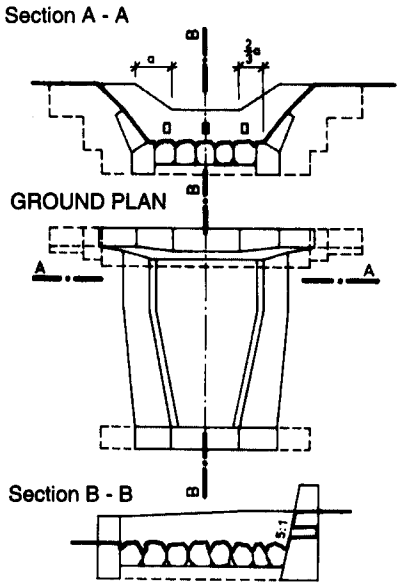


Fig.149 Stonework weir with an apron.

The stonework weirs should be large enough to ensure stability. Based on experience gained from design work, from standard projects and from the analysis of stable torrent regulation interventions, the recommended data on the width of the weir body at the spillway level are indicated in Table 66.

If a weir is to be designed for which the height data are not included in Table 68, a static calculation should first be performed to evaluate the stability. The axis of a stonework-masonry weir body is straight or it may be curved, usually circular, with a radius equal to the length of the chord (Fig. 150).

Concrete weirs. The design of concrete weirs should respect the same principles as with stone masonry. Since concrete is vulnerable to abrasion by the material carried by water, the spillway and the downstream face must be lined with quarry stone and cement mortar. The conditions for the use of concrete in building the weirs are the same as described with concrete check dams.

Table 66

Width in the crest of a weir body

Height of weir (m)	Depth of water in channel bed (m)		
	0.9–1.1	1.1–1.3	1.3–1.5
0.5–0.7	0.85–0.95	0.90–1.05	1.0 –1.15
0.7–0.9	0.90–1.0	0.95–1.1	1.05–1.20
0.9–1.1	0.90–1.0	1.0 –1.1	1.05–1.20
1.1–1.3	0.95–1.05	1.05–1.15	1.1 –1.25
1.3–1.5	0.95–1.1	1.05–1.15	1.15–1.25

Fig.150 Stonework weir with a curved axis (Photo V. Šmejc).

Weirs from precast concrete blocks. Prefabricated blocks are widely used in building torrent-control structures. The industrially produced prefab blocks are prismatic in shape and are furnished with holes by means of which more blocks can be fixed together. Such a structure is, in fact, a box consisting of two parallel precast block walls tied together by other precast parts laid across them. The prefabricated parts are put in place so that reinforcing bars can be stuck through the holes of the adjoining blocks; the holes are then filled with a thin concrete mix. This fixes the prefabricated blocks together and the structure gains strength. A concrete bed plate must have been prepared before the weir body of prefabricated parts is built. The space between the upstream and downstream walls of the structure is filled with refuse stone, rocks collected around the structure or poor-quality quarry stone. Quarry stone and cement mortar are used for the paving of the area between the prefabricated blocks at the crest of the wing walls and the spillway. The cross-section of the weir body is a rectangle; however, taller weirs need greater stability, so their upstream side is vertical and their downstream side retreats from the toe to the crest (the same as with check dams of prefab parts). The spillway is trapezoidal in shape and its slanting walls are step-like.

Wooden weirs. Wooden weirs (Fig. 151) are currently very exceptional in torrent control. The only places where timber is used are remote locations to which transport of material is difficult and local material has to be used.

The weir body consists of logs up to 0.3 m in diameter, hewn on contact surfaces, if necessary. The wooden wall is fixed to piles, up to 0.2 m in diameter, driven into solid ground to a depth of 1.5 m at a minimum. If the weir is high the logs are fixed to the piles with bolts; wrought-iron nails are used in smaller weirs. A sheet pile wall (piles up to 0.12 m thick) is built on the upstream side, otherwise water would scour the gravel under the weir and the apron.

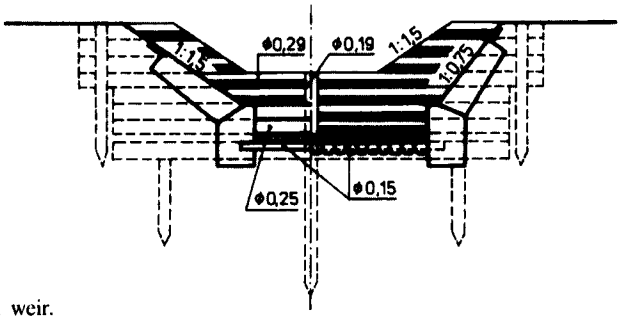


Fig.151 Wooden weir.

If such weirs are furnished with a stilling basin, its bottom is lined with stone paving laid onto a wooden grillage. The end sill consists of two logs fixed to piles.

Wooden weirs can only be built where piles can be driven into the channel bed. This condition is only met in torrents with fine-grained sediment.

Combined weirs. Wooden structures offer the advantage of easy and fast construction; the building material is locally available and, compared to masonry weirs, the wooden structures are flexible and more resistant to lateral pressures and to destruction if water leaks through them.

Unfortunately, wood is not a very durable material, especially in cases of frequent alternations of exposure to air and water. Combined weirs (Fig. 152) offset this disadvantage. The body of such a weir is wooden in the spillway part (where the wood always stays under water), whereas the remaining parts of the body consist of stonework masonry, using cement mortar. The wooden wall is held by piles in the middle part and its ends are sunk into recesses in the masonry. The remaining parts of the combined weirs are the same as in wooden weirs.

Stone-filled gabion weirs. Stone-filled gabion weirs are built on gravelly water courses where there is enough gravel to fill the cages and mattresses. The construction of stone-filled gabion structures is described in the section on check dams.

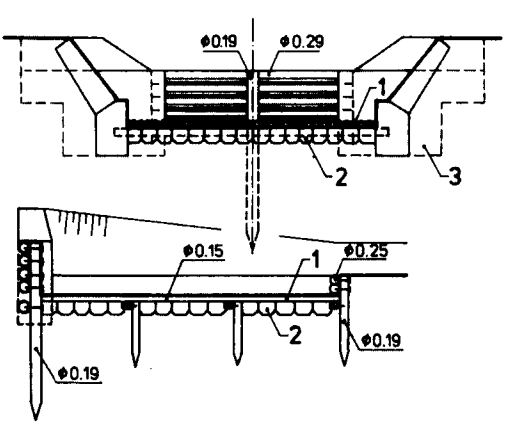


Fig.152 Combined weir: 1 – wooden floor, 2 – dry stone paving, 3 – stone masonry using cement mortar.

The downstream floor of the weir. Depending on the nature of the flow in the channel bed, the downstream floor may be protected either by an apron or a stilling basin. Their purpose is to dissipate the excess kinetic energy of the water falling down over the weir body. At the toe of the weir the stream floor is widened to safely cushion the fall of the overflowing nappe. Ideally, the width of the protected floor under the spillway should accommodate the whole width of the overfall. However, this requirement is not always easy to meet (it may be technically difficult or the costs may be too high). It is therefore recommended that the bottom of the apron or stilling basin should reach up to 2/3 of the projection of the sloping sides of the spillway (See Fig. 149). The protected floor tapers from the extended part at the weir's toe downstream to the original channel width. Stilling basins have an end sill at the end of the widened floor. The end sill's distance from the weir is determined by calculation. Aprons have stabilising belts at their downstream end which give them stability. As the length of the protected part of the downstream floor also depends on width, it is recommended to locate the place where the protected floor's width is reduced to the width of the channel bed farther downstream, behind the stabilising belt.

As said, the protected downstream floor should taper from the weir body towards the original channel profile. However, it is recommended that the widened part should be as long as the distance from the structure at which the overflowing nappe hits the floor; the tapering should start at that point.

Like the body of the weir itself, the end sill or the stabilisation belt must be keyed sufficiently deeply in the banks. To provide reliable support for the bottom of the apron or stilling basin, the end sill or stabilising belt foundation is 0.2 − 0.3 m under the bottom revetment.

The bottom of the stilling basin is horizontal and is lined so as to be able to absorb the pressure of the overflowing nappe.

The material most frequently used to line the bottom of the stilling basin is quarry stone paving. A concrete layer is first made, cement mortar is spread on its surface and the paving stones are embedded in the mortar. The thickness of the lining layer and the concrete foundation depends primarily on the height of the weir and ranges between 0.3 and 0.5 m for the pavement and 0.1 to 0.3 m for the concrete layer underneath. A baffle floor of pole-stages (0.12 − 0.13 m in diameter), nailed to round wood of the same diameter, laid across, may be installed on the stilling basin pavement to protect it against damage by the overflowing nappe; the round woods that hold the floor are fixed to piles and are embedded in the side walls (bank protection) of the stilling basin.

The stilling basins of smaller weirs can be protected by dry stone paving, the stones being laid into a wooden grillage which keeps the stones in place or, if damage occurs, reduces the damage to a minimum. It is also the case that dry stone paving can be protected by a wooden floor.

If the weir is furnished only with an apron and not with a stilling basin, the construction of the apron should provide sufficient resistance to the high velocities at the place of incidence of the overflowing nappe. The apron's surface should also have sufficient roughness to dissipate the excess kinetic energy of the overflowing nappe and to reduce the apron's length. Riprap is the best solution, and also stone packing with a very rough surface. The apron is either horizontal or may have the same inclination as the downstream channel bed.

The banks above an apron or stilling basin should be lined with a thick paving of quarry stones embedded in cement mortar: the thickness of the pavement layer should be 0.3 m at the minimum. Where the paving adjoins the body of the weir, it should have an inclination of 1:0.75; then, via a skew surface, it passes to the normal gradient of the bank slopes. At the toe of the slope the pavement is supported by a low longitudinal wall above the apron or stiling basin. The wall consists of quarry-stone masonry and cement mortar and its foundations are as deep as those of the end sill or the stabilising belt.

Sills and weirs: the hydraulic calculation

Transverse structures, built to adjust the slope of the channel bed, reduce the kinetic energy of the water current in torrents. However, on the transverse structures themselves the water's kinetic energy increases to reach a maximum at the place of incidence of the overflowing nappe. If no structural measure is taken to dissipate the excessive kinetic energy at that point, the current dissipates progressively in the channel bed downstream while the bottom and banks are scoured and sediment is washed away.

Under sills, weirs and check dams, the kinetic energy is cushioned by the roughness of the massive lining of the apron or by stilling basin.

If subcritical flow occurs in a torrent channel below the transverse structure, the transfer from supercritical flow to subcritical flow takes the form of a hydraulic jump in which the excess hydraulic energy is dissipated. Both a stilling basin and a rough-surfaced apron will serve well in such cases. The stilling basin is better for tall structures: its dimensions help to form a submerged hydraulic jump. For lower structures it is often more economical to build only an apron with a rough bottom in the nonuniform flow segment behind the place of incidence of the overflowing nappe and to "create" the unsubmerged hydraulic jump.

If there is supercritical flow in the channel downstream from the structure, the transition from the type of flow just below the structure to the channel flow takes place slowly within a comparatively long segment of nonuniform flow with a successive reduction of velocity. In such cases it is not recommended to build a stilling basin: the excessive kinetic energy (hence also the length of the high-velocity segment) can be reduced by an apron with a rough bottom.

Apron structure with subcritical flow. If an adjoining hydraulic jump occurs behind the place where the overflowing nappe hits the apron, the apron needs to be adequately lined just in the segment involved. Stone packing should be used to form the apron; the size of the stones should be adequate to withstand the velocity of the water at the place of incidence of the overflowing nappe.

With the nonsubmerged hydraulic jump, a nonuniform flow first develops behind the place where the overflowing nappe hits the apron and the water depth progressively increases up to the place where it equals the first conjugate water depth of the hydraulic jump $y_{1\,\mathrm{odd}}$ related to the depth of water in the channel y_d ($= y_{2\,\mathrm{odd}}$) (Fig. 153).

Then the length of the apron is

$$l = l_p + l_n + l_s \qquad \text{(m)}, \tag{6.114}$$

where l_p – the distance from the structure at which the overflowing nappe hits the apron (m),

l_n – length of the uniform supercritical flow segment (m),
l_s – the hydraulic jump (m).

The distance from the structure at which the overflowing nappe hits the apron, if there is subcritical flow in the upstream channel, is calculated from the equation

$$l_p = 2\sqrt{y_k(s + 0.5y_k)} \quad \text{(m)}, \tag{6.115}$$

where y_k – critical depth in the upstream channel (m),
s – the height of the weir structure (m).

It is also possible to use the equation

$$l_p = 1.64\sqrt{E_d(s + 0.24E_d)} \quad \text{(m)}. \tag{6.116}$$

The length of supercritical flow downstream from the place where the overflowing nappe hits the apron is determined from the nonuniform flow equation:

$$l_n = \frac{E_d - E_o}{i_o - i_E} \quad \text{(m)}, \tag{6.117}$$

where E_d – specific energy in the channel downstream from the structure (m),
E_o – specific energy in the place of incidence of the overflowing nappe (m),
i_o – downstream channel-bed slope,
i_E – energy line slope.

The energy slope between the place of incidence of the overflowing nappe and the first conjugate depth of the hydraulic jump is calculated from the equation

$$i_E = \frac{Q^2}{S_p^2 C_p^2 R_p}, \tag{6.118}$$

where Q – the given flow discharge ($\text{m}\cdot\text{s}^{-1}$),
S_p – average flow profile area of the supercritical flow segment,
C_p – average velocity coefficient of supercritical flow,
R_p – average hydraulic radius of the supercritical flow segment.

The depth of water flow in the place of incidence of the overflowing nappe y_o is calculated by iteration. The cross-sectional area S_o is calculated for the chosen depths and the velocity v_o is calculated from the continuity equation. The calculation is complete when at the selected y_o, the $E_o = E_d + s$.

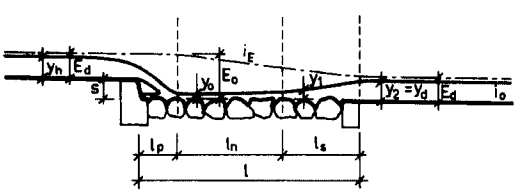


Fig.153 Reducing the kinetic energy on the apron at subcritical river flow in the channel.

The first conjugate depth of hydraulic jump $y_{1\,odd} = y_d$, where y_d is the depth of water in the channel downstream from the structure:

$$y_{1\,odd} = \frac{y_d}{2}\left(\sqrt{1 + \frac{8q^2}{gy_d^3}} - 1\right) \quad \text{(m)}, \tag{6.119}$$

where q – specific discharge ($m^3 \cdot s^{-1}$) (with trapezoidal cross-section it is related to the median diagonal).

As can be seen from equations (6.117) and (6.118), the length of the supercritical flow segment depends on the roughness of the wetted perimeter. It can be substantially reduced by increasing the roughness of the channel bed and banks, using stones of adequate size put into a stone packing.

KALIŠ (1973) derived the formula

$$n = \frac{\sqrt[6]{D}}{21.6} \tag{6.120}$$

for the degree of roughness of stone packing (stones spread as for pitching), where D is the longest dimension of the average stone (m).

The length of the nonsubmerged hydraulic jump can be determined from the following equations:

SMETANA:

$$l_s = 6(y_{2\,odd} - y_{1\,odd}) \quad \text{(m)} \tag{6.121}$$

NOVÁK:

$$l_s = K(y_{2\,odd} - y_{1\,odd}) \quad \text{(m)} \tag{6.122}$$

where:

$$K = 5.5 \quad \text{for} \quad 3 < \frac{y_2}{y_1} < 4$$

$$K = 5.0 \quad \text{for} \quad 4 < \frac{y_2}{y_1} < 6$$

$$K = 4.5 \quad \text{for} \quad 6 < \frac{y_2}{y_1} < 20$$

$$K = 4.0 \quad \text{for} \quad \frac{y_2}{y_1} > 20$$

Example 5

Weirs 0.5 m high were built on the torrent channel. The stable channel-bed slope between the weirs is $i = 0.008$. The channel of the torrent is trapeziform and has the following dimensions: $b = 6.0$ m, $y = 0.8$ m, bank slope gradient 1 : 1.5. The channel's average degree of roughness $n = 0.030$. Further data: $S = 5.44\ m^2$, $Q = 8.88$ m, $R = 0.613$ m, $v = 2.14\ m \cdot s^{-1}$, $Q = 10.06\ m^3 \cdot s^{-1}$.

a) Determining the distance from the structure at which the overflowing nappe hits the floor

$$l_p = 1.64\sqrt{E_d(s + 0.24E_d)},$$

$$E_d = y + \frac{v^2}{2g} = 0.8 + \frac{2.14}{19.62} = 1.03 \text{ m},$$

$$s = 0.5,$$

$$l_p = 1.64\sqrt{1.03(0.5 + 0.24 \cdot 1.03)} = 1.44 \text{ m}.$$

b) Determining the nonuniform flow segment

Nonuniform flow occurs between the place of incidence of the overflowing nappe and the place of the rise of the first conjugate depth of the hydraulic jump. To calculate this segment, it is first necessary to know the water depth at the place of incidence of the overflowing nappe y_o and the first conjugate depth $y_{1\,odd}$.

Water depth at the place of incidence of the overflowing nappe is determined by iteration using the following formula:

$$E_o = E_d + s = y_o + \frac{v_o^2}{2g} = 1.03 + 0.5 = 1.53 \text{ m}.$$

The cross-sectional area S_o, velocity of the water $v_o = \frac{Q}{S_o}$ and $E_o = y_o + \frac{v_o^2}{2g}$ are determined for the chosen water depth.

The calculation is completed when $E_o = y_o + \frac{v_o^2}{2g}$ is approximately equal to $E + s$. The procedure of the calculation is surveyed in Tab. 67.

The first conjugate depth of the hydraulic jump $y_{1\,odd}$ can be determined from the equation:

$$y_{1\,odd} = \frac{y_{2\,odd}}{2}\left(\sqrt{1 + \frac{8q^2}{g y_{2\,odd}^3}} - 1\right).$$

The second conjugate depth of the hydraulic jump $y_{2\,odd} = y_d$, i.e., the depth of water in the channel, hence $y_{2\,odd} = 0.8$ m. The specific discharge $q = \frac{Q}{b'}$, where b' is the median transversal of the trapezoidal channel:

$$b' = [(b + 2my) + b]0.5 = [(6.0 + 2 \cdot 1.5 \cdot 0.8) + 6.0]0.5 = 7.2 \text{ m}$$

$$q = \frac{Q}{b'} = \frac{10.06}{7.2} = 1.40 \text{ m} \cdot \text{s}^{-1}$$

$$y_{1\,odd} = \frac{0.8}{2}\left(\sqrt{1 + \frac{8 \cdot 1.40^2}{9.81 \cdot 0.8^3}} - 1\right) = 0.41 \text{ m}.$$

Table 67

Calculation of water depth at the site of incidence of overflowing nappe

y_0	S_0	$v_0 = \frac{Q}{S_0}$	$\frac{v_0^2}{2g}$	$E_0 = y_0 + \frac{v_0^2}{2g}$	$E_d + s$
0.4	3.16	3.18	0.52	0.91	1.53
0.3	2.35	4.28	0.93	1.23	
0.2	1.55	6.49	2.15	2.35	
0.28	2.19	4.59	1.08	1.36	
0.27	2.11	4.77	1.16	1.43	
0.26	2.03	4.96	1.25	1.51	1.53

The length of the nonuniform flow segments is

$$l_n = \frac{E_d - E_o}{i_o - i_E}.$$

The energy line slope between the cross section of y_o and $y_{1\,odd}$ will be determined from the equation:

$$i_E = \frac{Q^2}{S_p^2 C_p^2 R_p},$$

where S_p, C_p and R_p are average values of discharge area, velocity coefficient and hydraulic radius between profiles y_o and y_1 (Tab. 68).

Table 68

Calculation of the hydraulic characteristics for an average profile between y_0 and y_1

Cross section					
$y_0 = 0.26$ m		$y_1 = 0.41$ m		$y_p = 0.34$ m	
S_0	2.03 m^2	S_1	3.15 m^2	S_p	2.59 m^2
O_0	7.99 m	O_1	7.69 m	O_p	7.84 m
R_0	0.254 m	R_1	0.409 m	R_p	0.332 m
C_0	19.29	C_1	21.06	C_p	20.18

The velocity coefficient C is determined on the basis of the degree of roughness, using the formula

$$\frac{1}{n} = \frac{21.6}{D^{1/6}},$$

where D – the highest dimension of the riprap stone used to line the apron ($C = (1/n)R^{1/6}$):

$$\frac{1}{n} = \frac{21.6}{0.5^{1/6}} = 24.45.$$

The energy line slope is

$$i_E = \frac{10.06^2}{2.59^2 \cdot 20.18^2 \cdot 0.332} = 0.1116$$

The length of the nonuniform flow segment is

$$l_n = \frac{1.03 - 1.53}{0.008 - 0.1116} = 4.83 \text{ m}.$$

c) The length of the hydraulic jump
The length of the hydraulic jump is:

$$l_s = 6(y_2 - y_1) = 6(0.8 - 0.41) = 2.34 \text{ m}.$$

The total length of the apron is

$$l = l_p + l_n + l_s = 1.44 + 4.83 + 2.34 = 8.61 \text{ m}.$$

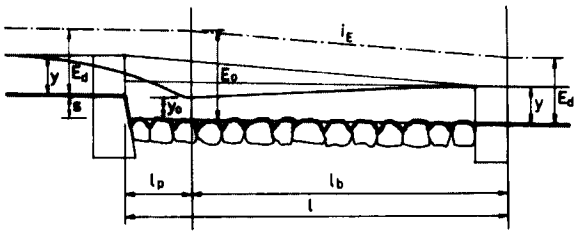


Fig.154 Reducing the kinetic energy on the apron under the weir at supercritical flow in the channel.

Apron structure with supercritical flow. With supercritical flow in the downstream channel, if there is no stilling basin to protect the floor (Fig. 154), cross waves develop under the weir or sill. The excess kinetic energy is substantially dissipated – hence the bottom is exposed to the greatest stress – in the place of incidence of the overflowing nappe and in the segment with cross wave motion. Hence, the nappe's clearance distance and the distance at which the cross waves meet determine the needed value of the apron's length:

$$l = l_p + l_b \qquad \text{(m)}. \tag{6.123}$$

The clearance distance of the overflowing nappe with supercritical flow in the upstream channel is determined from the equation

$$l_p = 2E_d\sqrt{(0.79s + 0.16E_d)} \qquad \text{(m)}. \tag{6.124}$$

LIBÝ (1973) performed model measurements from which he derived his equation for the maximum distance at which the cross waves meet

$$l_b = \frac{B_o}{2}\,\text{cotg}\,\beta \qquad \text{(m)}, \tag{6.125}$$

where B_o – width of overflowing nappe (m),
β – wave angle by which the longitudinal axis of the cross wave deflects from the original direction of the flow at an infinitely small change in depth.

The minimum value of $\sin\beta$ is calculated from the formula

$$\sin\beta = \frac{1}{Fr_o} \qquad \text{(m)}, \tag{6.126}$$

where Fr_o – Froud's number of the incident overflowing nappe at the place of incidence

$$Fr_o = \frac{v_o}{\sqrt{gy_o}}, \tag{6.127}$$

where v_o – overflowing nappe's velocity at its place of incidence ($m \cdot s^{-1}$),
y_p – depth at the place of incidence of the overflowing nappe (m).

The flow aerates while running along a very rough channel segment; owing to this, the length of the segment with cross wave motion can be reduced by 30%.

Example 6.

The weir is 0.5 m high. The stable slope of the channel bed is $i_d = 0.02$. The channel is of the same shape as in the case of the example with subcritical flow. Further data: $S = 5.44\ m^2$, $v = 3.4\ m \cdot s^{-1}$, $Q = 18.5\ m^3 \cdot s^{-1}$. Torrential flow arises in the channel.

d) Determining the clearance distance of the overflowing nappe:

$$l_p = 2E_d \sqrt{(0.79s + 0.16E_d)},$$

$$E_d = y + \frac{v^2}{2g} = 0.8 + \frac{3.4^2}{19.62} = 1.39\ m,$$

$$s = 0.5\ m,$$

$$l_p = 2 \cdot 1.39 \sqrt{(0.79 \cdot 0.5 + 0.16 \cdot 1.39)} = 2.18\ m.$$

e) Determining the distance of the centres of the cross waves

The first parameter to determine is the depth of water at the place of incidence of the overflowing nappe and the corresponding width at the water surface. The depth of water is determined in the same way as in the preceding example (i.e., with subcritical flow), using the equation

$$E_o = E_d + s = y_o + \frac{v_o^2}{2g} = 1.39 + 0.> = 1.89\ m.$$

The remaining part of the calculation is shown in Tab. 69.

At a depth $y_o = 0.44\ m$ the width of the water surface is

$$E_o = b_o + 2y_o m = 7.6 + 2 \cdot 0.44 \cdot 0.75 = 8.26\ m.$$

Further procedure:

$$Fr_o = \frac{v_o}{\sqrt{gy_o}} = \frac{5.3}{\sqrt{9.81 \cdot 0.44}} = 2.55,$$

$$\sin \beta = \frac{1}{Fr_o} = \frac{1}{2.55} = 0.39245,$$

Table 69

Calculation of the the depth of water at the site of incidence of overflowing nappe

y_0	S_0	$v_0 = \frac{Q}{S_0}$	$\frac{v_0^2}{2g}$	$E_0 = y_0 + \frac{v_0^2}{2g}$	$E_d + s$
0.3	2.35	7.87	3.15	3.45	1.89
0.4	3.16	5.85	1.75	2.15	
0.42	3.32	5.57	1.58	2.00	
0.43	3.41	5.43	1.50	1.93	
0.44	3.49	5.30	1.43	1.87	1.89

$$l_b = \frac{B_o}{2} \operatorname{cotg} \beta 0.7 = \frac{8.26}{2} \cdot 2.34 \cdot 0.7 = 6.76 \text{ m}.$$

The total length of the apron is

$$l = l_p + l_b = 2.18 + 6.76 = 8.94 \text{ m} = 9.0 \text{ m}.$$

The stilling basin. The stilling basin creates conditions for a hydraulic jump by which the excess kinetic energy of the overflowing nappe is dissipated (Fig. 155).

The greatest reduction of kinetic energy is attained with a submerged hydraulic jump at which the first conjugate depth of the hydraulic jump y_1 is smaller than depth y_o in the place of incidence of the overflowing nappe.

The parameters determined for the stilling basin include its depth and length.

The length of the stilling basin is the sum of the overflowing nappe's clearance and the length of the submerged hydraulic jump:

$$l_v = l_p + l_s \qquad \text{(m)}. \tag{6.128}$$

Methods described in hydraulics textbooks can be used for the determination of the depth of the stilling basin.

Rock chutes

The use of boulders to build chutes follows the model of steep torrent segments which developed in a natural way into boulder-bottomed rapids. The rock chutes are short segments of a steep torrent bed which are lined with large bare boulders (Fig. 156); they are designed to reduce the altitude difference between the upper and lower torrent channel bed. The first structures of this type were designed by RIEDIGER and SCHAUBERGER in Austria and were built in 1953 (SCHAUBERGER, 1957) and their use in torrent control has rapidly spread to other countries.

Part of the increased kinetic energy which arises as the torrent flow passes from the upstream to the downstream floor of the torrent is dissipated on the chute surface and the rest is dissipated in the hydraulic jump in the case of subcritical flow or on the roughened floor in the case of supercritical flow. The amount of kinetic energy dissipated depends on the roughness of the chute surface; hence, the surface of the chute should consist of boulders sticking out to different heights and increasing the energy loss by friction.

Rock chutes offer some advantages, compared to weir structures: they are of simple design, easy to build (no skilled labour is required and much of the work can be mechanised), locally available materials can be used, and the work can, if necessary, be done in winter. The time needed for construction is short and the

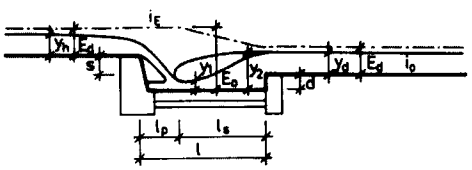


Fig.155 Dissipation of kinetic energy in stilling basin.

Fig.156 Adjustment of the channel bed slope using rock chutes (Photo K. Dimitrovský).

investment costs are usually lower than in the case of other types of transverse structures. Their advantages also include their natural appearance, that fits well into the environment, their contribution to the aeration of the water flow and the free passage of fish with no need for special fish-pass structures. The only disadvantage is that they require frequent maintenance interventions: removal of tree branches and other objects that get stuck between the boulders and addition of new boulders in places where the structure has been damaged.

Rock chutes can either be built as part of larger projects or as separate structures in local interventions, maintenance repairs or reconstructions. In some cases, e.g., in a deep channel bed between bank walls, it will suffice to build only the chute area.

The structural features of rock chutes include the crest, the chute area, chute bank protection, chute toe and protection of the downstream floor below the chute.

Straight stretches of torrent channels are the best locations for the construction of rock chutes. If the chutes are to be built in torrent bends, structural measures should be taken to maintain a uniform exposure of the chute area: the axis of the chute should be slightly rotated (10 − 12°) from the perpendicular line towards the tangent of the bend (Fig. 157). With shorter chutes it is advantageous to rotate the crest so that the axis of the chute passes through the points of intersection of the

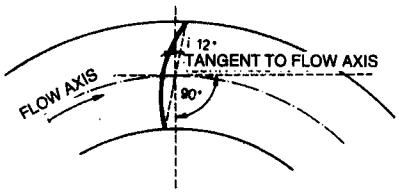


Fig.157 Location of a rock chute in a bend – theoretical principle.

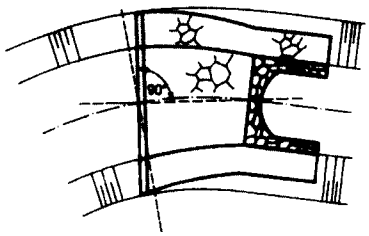


Fig.158 Location of a rock chute in a bend – practical realisation.

upper and lower edges of the chute area with the axis of the regulated channel (Fig. 158).

The crest. In rock chutes built within torrent control, the crest is flush with the level to the upstream channel bottom. The main reason why this is so is the hydraulic jump that develops – if the crest is above the upstream bottom – in the channel above the chute with supercritical flow which prevails in torrents. In fact, a taller crest is not even needed to protect the upper part of the chute because in a channel consisting of coarse-grained sediment the bottom does not deepen in front of the crest.

The crest of rock chutes may be either straight or arched upstream. The straight design prevails in torrents; only in torrents whose bottom is wider than 15 m is it possible to build an arched crest. Arched structures concentrate much of the flow in the centre of the channel, the chute structure is exposed to intensive stress and deep scours develop in the downstream floor: this may endanger the stability of the revetment of channel banks. A curved spillway crest is also unsuitable for chutes taller than 3 m.

Arched crests may be designed as compound arches with the central part's radius equal to the width of the channel, the radius of the lateral parts (near the banks) being double that of the central part. They may also be designed as simple arches with a radius of $1.25B$. ZÁSTĚRA et al. (1984) recommend that the rise of the crest arch should not be greater than $0.1B$ in chutes up to 2 m in height and $0.05B$ in those above 2 m.

A sheet pile wall, a log wall or a concrete belt may be built along the crest of the chute. Such a wall is not necessary for the stability of the chute but it will prevent water from penetrating under the stone packing and removing the smaller particles from under the chute area. At low discharges the wall forces the water to run on the stone packing on the surface of the chute and not under it. For this purpose, it is recommended to use the same rocks to build both the chute itself and the crest: in the crest the stones should be fixed in concrete with their tops being left to project up to $1/3D$ so as not to reduce the roughness of the surface (ZÁSTĚRA et al., 1984).

The chute area. The design of the chute area is closely related with that of the crest. If the crest is straight, the chute surface is dish- or shell-shaped.

The main factors to be considered when designing the slope of the chute area include the formation of the channel at the site where the rock chute is located, the extent to which the boulders can be fixed and stabilised in the chute area, the required dissipation of kinetic energy and, of course, the economic requirements.

Kinetic energy is best dissipated on slightly inclined chute areas which also have the greatest stability. However, such areas are not very advantageous from the economic point of view. Very steep chutes are less effective in reducing the kinetic energy of water and the boulders on the chute area have to be heavy. Considering all the aspects involved, the recommended slopes are 1:6 + 1:12, the optimum being 1:10.

Chutes do not have a uniform slope – on the contrary, a steeper slope below the crest and a milder slope farther downstream are better from the point of view of both the location of the structure in the controlled torrent segment and the reduction of the stream's kinetic energy.

To gain roughness, the chute surface is lined with stone packing of large boulders laid flat close to each other. To fix the boulders, smaller stones and gravel should be used to fill the gaps; this will help to maintain the water on the chute surface even at low flow rates.

The flow profile of rock chutes with a plane chute surface is a simple trapezoid. The banks are lined with stone packing, the rocks being of the same size as in the chute. For better inclusion in the landscape, it is recommended to plant the part of the slope above the stone packing with deciduous woody species which will be incorporated in the bank vegetation.

The toe of the chute. The toe of the chute is a place where the chute area is exposed to the greatest danger. If it is clear that the chute area has sufficient stability in itself, there is no need to install any extra stabilising elements at the chute toe, but where the stability is not guaranteed, it is recommended either to double the last several rows of boulders or to pour concrete between the boulders at the chute's toe. Sheet piling is not recommended: if a scour develops at the toe of the chute, the water will fall over the sheet piling, deepening the scour and cutting the bottom closer to the chute toe, thus endangering the chute's stability.

Protection of the downstream floor. The torrent bottom at the downstream end of the chute has to be protected if the need of such protection has been demonstrated by calculation. Stone packing is used for this purpose, the size of the stones being the same as on the chute. If the segment that has to be protected is long, the rocks placed farther downstream may be smaller. In torrents the length of the segment in which the whole width of the bottom has to be protected is determined by calculation; in torrents whose bottom consists of small-grained sediment, the protected segment may be extended by riprap spread along the banks to improve the stability of the channel banks (Fig. 158).

It is also possible to build a stilling basin below the chute to dissipate the excessive kinetic energy of supercritical flow. The depth and length of the stilling basin are determined by hydrotechnical calculation. Such a stilling basin is built as an extension of the chute area below the level of the downstream floor. The bottom of the basin is flat and returns to the level of the downstream floor at an inclination of 1:15 + 1:20.

Hydraulic computation for rock chutes

The factors that have to be monitored and measured include the hydraulic conditions of the flow in the channel upstream and downstream from a chute and their effect on the stability of the chute area and on the stability of the bottom at the site where the water levels meet below the chute. The chute's basic parameters,

i.e., its height and slope, are derived from the design parameters of the over-all project for the regulation of the longitudinal slope of the stream channel bed. Together with the design discharge and with the form and dimensions of the chute's cross section, these parameters provide the input data for the hydraulic solutions that have to be prepared with due respect to the nature of flow in the channel both upstream and downstream.

Subcritical flow in the upstream and downstream channel

Designing the chute area. The uniform flow in the upstream channel (depth y) passes above the crest of the chute over the critical depth of y_k to nonuniform flow on the chute; the flow may be nonuniform either in a short segment of the chute or over its whole length.

Uniform flow on a rock chute is characterised by the basic hydraulic parameters y_o, R_o, v_o and E_o which are determined when the stage-discharge relationship is calculated. The average channel velocity on the chute area with uniform motion v_o is calculated using the following roughness levels (KALIŠ, 1973):

for stones laid in packed form:
$$n = \frac{D^{1/6}}{21.6}, \tag{6.129}$$

for stones laid loosely:
$$n = \frac{D^{1/6}}{24.4}. \tag{6.130}$$

Data from Tab. 70 compiled by ZÁSTĚRA et al. (1984) for different specific discharge rates, chute heights and chute area slopes can be used for the determination of the size of the stones. As rock chutes built for torrent control have no freeboard above the upper channel bottom, the specific discharge rate can be calculated from the equation

$$q = \frac{Q}{b + my} \qquad (\mathrm{m^2 \cdot s^{-1}}). \tag{6.131}$$

The factor determining whether uniform flow develops on the chute area is the length of the nonuniform flow segment, measured from the crest. According to Fig. 159, the length of the nonuniform segment between the profile at the chute's crest (where the depth is equal to the critical depth y_k) and profile O at the start

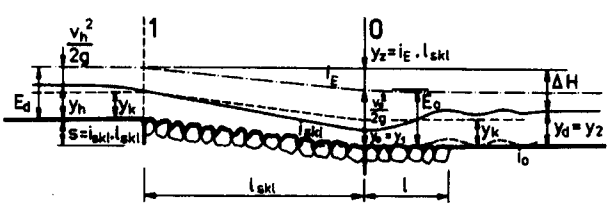


Fig.159 Flow on the chute with subcritical flow in the channel both upstream and downstream from the chute.

of uniform flow (where the depth is y_o) will be determined from the equation

$$l_n = \frac{E_o - E_d}{i_{skl} - i_E} \quad \text{(m)}. \tag{6.132}$$

The critical depth y_k on the crest for a trapezoidal profile is determined from the critical depth for a rectangular profile:

$$y_{lk} = \left(1 - \frac{\sigma}{3} + 0.105\sigma^2\right) y_{ok}, \tag{6.133}$$

where σ – parameter expressing the effect of profile shape

$$\sigma = \frac{my_{ok}}{b + my}, \tag{6.134}$$

y_{ok} – critical depth of a rectangular profile

$$y_{ok} = \sqrt[3]{\frac{\alpha q^2}{g}}. \tag{6.135}$$

To solve equation (6.132) it is necessary to determine the hydraulic characteristics S, O, R, v, C and E_d in the profile at the crest and in profile 0 on the assumption that

$$v = \frac{Q}{S}, \qquad C = \frac{v}{\sqrt{Ri}} \qquad \text{and} \qquad E_d = y + \frac{v^2}{2g}.$$

The energy line slope between these profiles will be determined for the average values of S, C and R, using the equation

$$i_E = \frac{Q^2}{S_p^2 C_p^2 R_p}. \tag{6.136}$$

If the nonuniform flow segment l_n is longer than the length of the chute area, it is necessary to determine the depth y_1 and other hydraulic parameters for the profile at the toe of the chute. The depth at the toe of the chute is determined by iteration to meet the condition that $E_1 = y_1 + v_1^2/2g$. The specific energy E_1 is calculated from the equation

$$E_1 = E_d + l_{skl}(i_{skl} - i_E). \tag{6.137}$$

The stability of the chute area. The surface of a rock chute is exposed to the greatest stress in the part of the profile in which the greatest cross-sectional velocity arises. If there is uniform flow over the chute area, the whole segment involved is exposed to the greatest stress. If there is nonuniform flow over the whole length of the chute, the highest velocity occurs at the toe of the chute.

Flow taking place over the chute area produces shear stress on the surface of the stone packing

$$\tau_o = \varrho g y i_E \qquad \text{(Pa)}. \tag{6.138}$$

If stability is to be attained in the stone packing on the chute area, the shear stress must not be higher than the critical shear stress

$$\tau_c = Kg(\varrho_m - \varrho) D C_s \qquad \text{(Pa)}, \tag{6.139}$$

where K – value experimentally determined in model trials (KALIŠ, 1971); $K = 0.047$,

ϱ_m – density of the rock from which the stones for the stone packing were taken; the average value is $\varrho_m = 2650\ \text{kg}\cdot\text{m}^{-3}$,

D – the largest dimension of the average stone (m),

C_s – coefficient expressing the effect of the chute slope and the boulder material's internal friction in water on the stability of the stone packing

$$C_s = \frac{\text{tg}\,\varphi - \text{tg}\,\alpha}{\text{tg}\,\varphi},$$

where φ – angle of the internal friction of stone packing in water; $\varphi = 41°$,

α – angle of slope of the stone surface; $\text{tg}\,\alpha = i_{skl}$.

Using the equation of equilibrium between actual and critical shear stress

$$\varrho g y i_E = Kg(\varrho_m - \varrho) D C_s, \tag{6.140}$$

where $\frac{v^2}{C^2 R}$ is substituted for i_E and $\frac{24.4}{D^{1/6}}$. $R^{1/6}$ is substituted for C, and assuming that $R = y$, the equation for critical velocity is obtained:

$$v_v = 6.8 D^{1/3} y^{1/6} C_s^{1/2} \qquad (\text{m}\cdot\text{s}^{-1}). \tag{6.141}$$

Stability of the stone packing on the chute surface is achieved if the critical velocity $v_v \geqq v$. The required size of the stones can be derived from this condition:

$$D = \frac{0.003 v^3}{y^{1/2} C_s^{3/2}} \qquad \text{(m)}. \tag{6.142}$$

The velocity that always occurs at the toe of the chute with both uniform and nonuniform flow is substituted for velocity v in equation (6.142).

Hydraulic solution for the downstream floor. Since supercritical flow develops on the chute surface, the upstream water level is connected with that downstream through a hydraulic jump. The first conjugate depth of hydraulic jump is the depth y_1 at the toe of the chute. The other conjugate depth y_2 can be more or less accurately calculated from the equation

$$y_2 = \frac{y_1}{2}\left(\sqrt{1 + \frac{8q^2}{g y_1^3}} - 1\right) \qquad \text{(m)}. \tag{6.143}$$

The length of the hydraulic jump is determined from the simple hydraulic-jump equation

$$l_s = 6(y_2 - y_1) \qquad \text{(m)}. \tag{6.144}$$

The channel floor in the segment with hydraulic jump is exposed to a high stress, hence the floor must be protected by stone packing, the stones being of the same size as the rocks on the chute area.

The location of the site where hydraulic jump arises has to be identified if it is to be decided whether the downstream floor (below the chute) has to be protected. The key parameter is the second conjugate depth y_2: if $y_2 = y_d$ the hydraulic jump develops at the chute's toe and the bottom has to be protected over the whole length of the hydraulic jump. If $y_2 \geqq y_d$ it will be necessary to build a stilling basin at the toe of the chute to dissipate the excessive kinetic energy. KNAUSS (1977) recommends the following dimensions for such stilling basins:

length: $$l_v = 5.5(y_2 - y_1), \tag{6.145}$$

depth: $$d = 0.5D. \tag{6.146}$$

To attain stability on the downstream floor at the toe of the chute, it is most advantageous to have a $y_2 < y_d$. Under such conditions a backwater hydraulic jump arises and progresses upstream above the chute. If the whole hydraulic jump remains upstream from the chute, the downstream floor need not be protected.

Supercritical flow in both the upstream and downstream channels

At supercritical flow the depth over the crest is the same as in the channel. The solutions applying to the flow upstream from the chute and to the stability of the chute surface are the same as with the supercritical type of flow.

No hydraulic jump occurs at the transition from supercritical flow in the chute area to supercritical flow below the chute. The effect of the high degree of roughness of the stone packing on the downstream floor (stones of the same size as on the chute area) can be used to dissipate the excessive kinetic energy below the toe of the chute. The needed length of the protected downstream floor segment can be calculated from Libý's equation (Libý, 1973):

$$l = \frac{B}{2} \operatorname{cotg} \beta \qquad \text{(m)}, \tag{6.147}$$

where B – the width of water level at the toe of the chute (m),
β – angle at which cross waves meet;

$$\sin \beta = \frac{1}{F_r}$$

$$F_r = \frac{v_1}{\sqrt{g y_1}}.$$

Example 7

1. *Solution for rock chute with subcritical type of flow*

The height of the rock chute is 0.8 m, the slope of the chute area is 1:10. The design channel parameters upstream and downstream from the chute are as follows: bottom width $b = 6.0$ m, depth $H = 1.0$ m, gradient of the channel banks 1:1.5, channel bed slope $i = 0.008$, discharge $Q = 19.0\ \mathrm{m^3 \cdot s^{-1}}$. Hydraulic characteristics of the channel cross section:

$$S = 7.5\ \mathrm{m^2}, \quad O = 9.61\ \mathrm{m}, \quad R = 0.78\ \mathrm{m}, \quad v = 2.53\ \mathrm{m \cdot s^{-1}}$$

specific discharge:

$$q = \frac{Q}{b + my} = \frac{19.0}{6 + 1.5 \cdot 1.0} = 2.53\ \mathrm{m^2 \cdot s^{-1}},$$

critical depth:

$$y_{ok} = \sqrt[3]{\frac{aq^2}{g}} = \sqrt[3]{\frac{1.1 \cdot 2.53}{9.81}} = 0.90\mathrm{m}$$

$$\sigma = \frac{my_{ok}}{b + my} = \frac{1.5 \cdot 0.90}{6 + 1.5 \cdot 1.0} = 0.18$$

$$y_{lk} = \left(1 - \frac{\sigma}{3} + 0.105\sigma^2\right) y_{ok} = \left(1 - \frac{0.18}{3} + 0.105 \cdot 0.18^2\right) 0.9 = 0.85\ \mathrm{m};$$

further data on the profile at the crest:

$$S = (6.0 + 1.5 \cdot 0.85)0.85 = 6.18\ \mathrm{m^2}$$

$$Q = 6 + 2 \cdot 1.53 = 9.06\ \mathrm{m} \qquad R = 0.68\ \mathrm{m}$$

$$v = \frac{Q}{S} = \frac{19.0}{6.18} = 3.07\ \mathrm{m \cdot s^{-1}}$$

$$E_d = 0.85 + \frac{3.07^2}{2 \cdot 9.81} = 1.33\ \mathrm{m}$$

$$C = \frac{v}{\sqrt{Ri}} = \frac{3.07}{\sqrt{0.68 \cdot 0.008}} = 41.62\ \mathrm{m^{0.5} \cdot s^{-1}},$$

uniform flow on the chute area:

From Tab. 70, $q = 2.53\ \mathrm{m^2 \cdot s^{-1}}$. Stone size $D = 0.6$ m corresponds to the chute height of 0.8 m;

$$n = \frac{D^{1/6}}{24.4} = \frac{0.6^{1/6}}{24.4} = 0.038,$$

$$\frac{1}{n} = \frac{1}{0.038} = 26.32.$$

The following hydraulic values correspond to the discharge rate of $Q = 19.0\ \mathrm{m^3 \cdot s^{-1}}$:

$$y_o = 0.55\ \mathrm{m} \qquad S_0 = 3.75\ \mathrm{m^2} \qquad O_o = 7.98\ \mathrm{m} \qquad R_o = 0.47\ \mathrm{m}$$

$$v_o = 5.03\ \mathrm{m \cdot s^{-1}} \qquad E_0 = 1.83\ \mathrm{m} \qquad Q = 18.86\ \mathrm{m^3 . s^{-1}}$$

Table 70

Greatest heights of chute

Largest dimension Diameter of stone D (m)	Stone weight kg	Specific discharge q ($m^2 \cdot s^{-1}$)	Chute gradient			
			1 : 6	1 : 8	1 : 10	1 : 12
0.3	10	1	0.25	0.32	0.48	0.64
		2	0.18	0.24	0.28	0.33
		3	0.17	0.21	0.24	0.27
		4	0.16	0.20	0.23	0.26
		5	0.12	0.15	0.18	0.22
0.4	23	1	0.41	0.73	1.60	∞
		2	0.28	0.36	0.46	0.60
		3	0.25	0.31	0.37	0.44
		4	0.23	0.30	0.35	0.39
		5	0.20	0.25	0.29	0.34
0.5	46	1	0.67	∞	∞	∞
		2	0.41	0.55	0.84	1.6
		3	0.35	0.45	0.55	0.70
		4	0.32	0.40	0.49	0.58
		5	0.29	0.37	0.43	0.50
0.6	81	1	1.09	∞	∞	∞
		2	0.56	0.88	1.72	∞
		3	0.46	0.60	0.84	1.16
		4	0.42	0.52	0.65	0.82
		5	0.38	0.49	0.57	0.70
0.7	130	1	∞	∞	∞	∞
		2	0.75	1.56	∞	∞
		3	0.60	0.82	1.28	∞
		4	0.53	0.68	0.92	1.2
		5	0.48	0.62	0.76	0.96

$$C_o = \frac{v_o}{\sqrt{R_o i_{skl}}} = \frac{5.03}{\sqrt{0.47 \cdot 0.1}} = 23.20 \text{ m}^{0.5} \cdot \text{s}^{-1}$$

$$S_p = (6.18 + 3.75)0.5 = 4.97 \text{ m}^2$$

$$R_p = (0.68 + 0.47)0.5 = 0.58 \text{ m}$$

$$C_p = (41.62 + 23.20)0.5 = 32.41 \text{ m}^{0.5} \cdot \text{s}^{-1}$$

$$i_E = \frac{Q^2}{S_p^2 C_p^2 R_p} = \frac{18.86^2}{4.97^2 \cdot 32.41^2 \cdot 0.58} = 0.0236$$

the length of the segment with nonuniform flow:

$$l_n = \frac{E_o - E_d}{i_{skl} - i_E} = \frac{1.83 - 1.33}{0.1 - 0.0236} = 6.54\ \text{m} < l_{skl} = 8.0\ \text{m};\quad \text{i.e.,}$$

uniform flow arises on the chute area;
evaluation of stability of the chute area:

$$D = \frac{0.003 v_o^3}{y_o^{1/2} C_s^{3/2}} = \frac{0.003 \cdot 5.03^3}{0.55^{0.5} \cdot 0.885^{1.5}} = 0.62\ \text{m} > 0.60\ \text{m},\quad \text{i.e.,}$$

the design stone size provides stability of the chute area.

Designing the protection of the downstream floor:
the first conjugate depth of hydraulic jump $y_1 = y_0 = 0.55$ m,
the other conjugate depth of hydraulic jump y_2:

$$y_2 = \frac{y_1}{2}\left(\sqrt{1 + \frac{8q^2}{g y_1^3}} - 1\right) = \frac{0.55}{2}\left(\sqrt{1 + \frac{8 \cdot 2.53^2}{9.81 \cdot 0.55^3}} - 1\right) = 1.29\ \text{m},$$

$y_2 = 1.29\ \text{m} > 1.0 = y_d \ldots$ a stilling basin is designed, having a length $l_v = 5.5(y_2 - y_1) = 5.5(1.29 - 0.55) = 4.07$ m and a depth $d = 0.5D = 0.5 \cdot 0.6 = 0.3$ m.

2. *Solution for rock chute with supercritical flow*

The shape and dimensions of the channel cross section are the same as in the preceding example. The height of the chute and slope of the chute area are also the same ($s = 0.8$ m, $i_{skl} = 0.1$). The slope of both the upstream and downstream floors is $i = 0.018$.
Hydraulic characteristics of the channel:

$$S = 7.5\ \text{m}^2 \qquad O = 9.61\ \text{m} \qquad R = 0.78\ \text{m} \qquad v = 3.79\ \text{m}\cdot\text{s}^{-1}$$

$$C = 31.99\ \text{m}^{0.5}\cdot\text{s}^{-1} \qquad Q = 28.43\ \text{m}^3\cdot\text{s}^{-1} \qquad E_d = 1.73\ \text{m},$$

specific discharge:

$$q = \frac{Q}{b + my} = \frac{28.32}{6.0 + 1.5 \cdot 1.0} = 3.79\ \text{m}^2\cdot\text{s}^{-1},$$

uniform flow on chute area:

From Tab. 70, $q = 3.79\ \text{m}^2\cdot\text{s}^{-1}$; stone size $D = 0.7$ m corresponds to the chute height of 0.8 m;

$$n = \frac{D^{1/6}}{24.4} = \frac{0.7^{1/6}}{24.4} = 0.039 \quad \frac{1}{n} = \frac{1}{0.039} = 25.64.$$

The following hydraulic data correspond to the discharge of $28.43\ \text{m}^3\cdot\text{s}^{-1}$:

$$y_o = 0.71\ \text{m} \qquad S_o = 5.01\ \text{m}^2 \qquad O_o = 8.56\ \text{m} \qquad R_o = 0.59\ \text{m}$$

$$v_o = 5.70\ \text{m}\cdot\text{s}^{-1} \qquad E_o = 2.37\ \text{m} \qquad Q = 28.56\ \text{m}^3\cdot\text{s}^{-1}$$

$$C_o = 23.48\ \text{m}^{0.5}\cdot\text{s}^{-1}$$

$$S_p = (7.5 + 5.01)0.5 = 6.23\ \text{m}^2$$

$$R_p = (0.78 + 0.59)0.5 = 0.69\ \text{m}$$

$$C_p = (31.99 + 23.48)0.5 = 27.74\ \text{m}^{0.5}\cdot\text{s}^{-1}$$

$$i_E = \frac{28.56^2}{6.23^2 \cdot 27.74^2 \cdot 0.69} = 0.0396,$$

the length of the segment with nonuniform flow:

$$l_n = \frac{E_o - E_d}{i_{skl} - i_E} = \frac{2.37 - 1.73}{0.1 - 0.0396} = 10.60 \text{ m} > 3.0 \text{ m} = l_{skl}; \quad \text{i.e.,}$$

no uniform flow arises in the chute area.

Determining the hydraulic characteristics at the toe of the chute:

$$E_1 = E_d + l_{skl}(i_{skl} - i_E) = 1.73 + 8.0(0.1 - 0.0396) = 2.21 \text{ m}$$

$$y_1 = 0.75 \text{ m} \qquad S_1 = 5.34 \text{ m}^2 \qquad v_1 = Q/S_1 = 28.56/5.34 = 5.35 \text{ m} \cdot \text{s}^{-1}.$$

Evaluating the stability of the chute area:

$$D = \frac{0.003 v_1^3}{y_1^{0.5} C_s^{1.5}} = \frac{0.003 \cdot 5.35^3}{0.75^{0.5} \cdot 0.885^{1.5}} = 0.64 \text{ m} < 0.7 \text{ m}.$$

The design stone size is larger than the calculated size; the chute surface has stability.

Designing the downstream floor protection:

$$Fr_1 = \frac{v_1}{\sqrt{y_1 g}} = \frac{5.35}{\sqrt{0.75 \cdot 9.81}} = 1.961$$

$$\sin \beta = \frac{1}{Fr_1} = \frac{1}{1.961} = 0.5099 \qquad \text{cotg } \beta = 1.687$$

$$B_o = b + 2y_1 m = 6.0 + 2 \cdot 0.75 \cdot 1.5 = 8.25 \text{ m},$$

the length of the protected segment of the downstream floor:

$$l = 0.5 B_o \text{ cotg } \beta = 0.5 \cdot 8.25 \cdot 1.687 = 6.96 \approx 7.0 \text{ m}$$

Sediment catching structures — check dams

Check dams (Fig. 160) are transverse structures having retention space for the interception of the material which the water carries in large quantities downstream from the nonregulated upper stretches of the torrent in periods of storm flows. Depending on the purpose for which the sediment is trapped, the check dams are divided into retention dams and consolidation dams.

Retention dams

Retention dams are designed to check the transport of sediment further downstream and to prevent, in this way, the formation of a gravelly bottom in the downstream channel and deposition of sediment in the fields along the torrent, inside villages, near various structures, etc. Check dams should always be built at the upstream end of larger regulated torrent stretches; if sediment is not checked, the material will clog the channel in the regulated segment in which the bed slope was adjusted (reduced) to a stabilised level. On torrents that are not regulated, the check dams are usually built above residential areas, on gravel-bearing tributaries above their confluence with the main stream or at the downstream end of deep valleys.

The design of any retention dam should be based on the calculation of the expected amount of sediment carried as bottom load in the torrent [equations (6.40; 6.46; 6.47; 6.58 and 6.63)]. The required dam height, the location of the dam and the corresponding optimum kind of construction will be identified from

Fig.160 A stonework dam on the Bílá Desná in the Jizerské Mts. (Photo L. Novák).

the sediment data, which serve as a basis for calculating the needed volume of the catch basin above the dam.

The purpose of retention dams is to intercept only the flood flow of sediment which would otherwise cause damage on the lower course of the torrent. The normal flow of fine suspended load should not be interrupted: if all sediments were intercepted by the check dam, the catch basin would soon be filled and the dam would no longer serve its purpose.

The main factor to be considered when choosing the best place for a retention dam is the need to attain the required sediment interception effect at minimum construction costs. The economic aspects of the profile are evaluated from the amount of sediment retained per 1 m^3 of the volume of the dam body or per unit investment cost. Narrow profiles above which the valley spreads to form the needed catch basin are the best locations for retention dams.

Selection of the type of dam construction depends on the main characteristics of the structure site profile, including the profile's width and depth, the mechanical properties of the rocks and earths forming the bottom and banks, the stability of the slopes and other parameters. If an optimum type of retention dam structure is to be chosen as the best option from the economic and engineering point of view, the structure site should be subjected to detailed investigation.

The assumed amount of sediment carried by a stream can be intercepted either by a single dam or (more frequently) by a set of dams built in the valley of the torrent. The estimate of the total amount of sediment carried to the catchment basin does not fully express the total effect of all factors that may be involved, so it often happens that further sediment is carried to the site although the dams' catchment basin is already full. In such cases, the gravel deposits may be removed to empty the catchment basin; other solutions are to raise the height of the dam

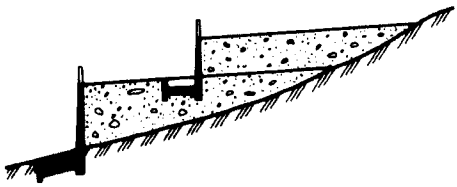


Fig.161 Enlarging the catchment space by building another dam on the deposited sediment.

body or to build new dams farther upstream. The dam body can be raised if both the longitudinal and crosswise profiles of the valley provide potential for enlarging the catchment space and if the foundations and the body of the original dam are strong enough. If conditions are favourable, another retaining dam can be built in a suitable profile on the sediment deposited in the already filled catchment area (Fig. 161).

Consolidation dams

The purpose of consolidation dams is to retain sediment in the catchment space, to fill the channel, to check mass flow of sediment and prevent it from moving further downstream and, if necessary, to provide support to deeply scoured or slided banks.

Consolidation dams are only designed to stabilise the devastated stretches of torrents if the deepened bottom does not need to be raised immediately. The dams' catchment basins are filled with sediment successively and a comparatively long time elapses before they are completely filled. If a stable channel has to be attained immediately, weirs are built to regulate the gradient.

A torrent segment damaged by scouring can be protected either by building one or several tall structures or a larger number of low check dams (Fig. 162).

The height of the structure depends on the extent to which the channel was deepened and on the length of the devastated segment. The "one big structure" approach is used when a short and deep scour is to be stabilised or when a slope damaged by landslide has to be protected. A greater number of lower dams are built to stabilise longer damaged stretches, especially in those cases in which the scours are not so deep. In such cases, the consolidation dams are intended to catch sufficient sediment to create a new bottom. Small dams are more suitable for this purpose and their construction is less expensive than building tall structures since

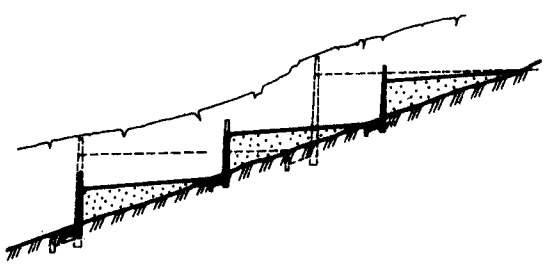


Fig.162 Comparison of the effectiveness of high and low consolidation dams.

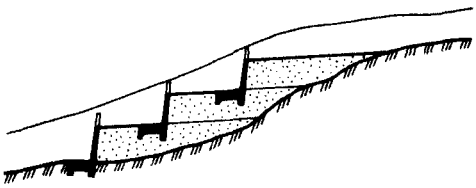


Fig.163 Filling a deep scour using a series of dams.

the latter are labour-intensive, need more building material and their maintenance is costly.

The channel bed slope between the low dams will be steeper: the lower the dam the narrower the channel, and the narrower the channel the higher the carrying force of the stream, leaving only larger-grained bottom load to settle.

Sediment is deposited above the check-dam body at a slope corresponding to the equilibrium slope of the sediment mix. This slope is much lower than that of the deepened bottom before the dam was built: it controls the distance of the dam's effect. Hence, if only one consolidation structure is built, the deposit should stretch over the whole devastated segment; if a larger number of smaller dams is built, the sediment deposit above each dam should reach upstream up to the toe (apron) of the next dam because the whole regulated segment has to be protected against bottom erosion. The only exception are segments in which a rocky bottom is exposed near the toe of the next upstream structure.

Floods may sometimes cut short but deep scours which cannot be removed by building just a single dam. In such cases several dams have to be built in succession: when the first dam's catchment basin is filled another dam is built straight on its sediment deposit (Fig. 163).

When preparing a project of regulation using consolidation dams (choosing the site, determing the dams' height and type of construction), local conditions must be taken into account, including the extent of the deepening of the bottom, the condition of the bottom and banks, the possibility of attaining a long-standing effect by building a low structure, etc. When the location of consolidation dams is considered, it must be known whether a new channel is to be built after the catchment basins are filled and consolidation is attained: if there is to be a new channel, the issue of the future channel's alignment must be solved while designing the dams and the dams have to be designed so as to correspond to the future flow conditions.

If there are large sediment deposits (left in the torrent channel from a period of a low carrying force of the water or brought down by a devastated tributary, or coming from a landslide on the bank), a dam has to be built to prevent future floods from carrying such deposits away.

Consolidation dams are also built in places where a deep scour in the bottom has affected the stability of the slopes or where the slopes were broken by a landslide. In such cases, only one structure is built: it must be sufficiently high to enable the formation of a deposit which would support the toe of the damaged slopes and also to enable the adjustment of the slopes to a gradient corresponding to the natural slope of the earth material.

Dam construction

The construction of dams often involves high costs, so efforts are always taken to find new design types to provide the required performance at the lowest possible cost. Preference is given to construction types allowing the use of machines, to adjusting the construction technique to the local conditions of sites outside easily accessible areas, to using new building materials and to saving the costs of construction and maintenance.

Dams of all design types have a dam body and a protected downstream floor. The dam body has a spillway in its upper part, dimensioned to accommodate the design flow. When designing the spillway it is always necessary to consider the effects of a 100-year flood.

The spillway is located in the middle part of the dam body; on both sides it passes into the wing walls keying the dam into the banks. The foundation must be deep so as not to allow water to cut scours between the dam and the bank, thus "bypassing" the dam. Holes corresponding to the sizes of the sediment mix in the catchment basin are left in the dam body to drain water from the catchment basin and to allow the smaller particles to be carried further downstream (thereby not unnecessarily reducing the available sediment accumulation space).

The protected floor below the dam is of similar design and has similar dimensions to that built below weirs. It is usually possible to build only an apron below the dam: with an apron the dam body does not need to be keyed so deeply as with a stilling basin.

Masonry dams. Stonework masonry dams are the most common structural type of dams. The stones are laid either dry or using cement mortar. Quarry stone is normally used but in inaccessible places the rocks may be collected at and around the building site. The ground plan of a masonry dam may be either straight or arched.

The body of the straight masonry dams is trapezoidal in cross-section, its upstream face being vertical and the downstream side having an inclination of 5:1. In dams whose body transmits the load of its own mass as a cantilever, the downstream side usually has a smaller gradient, or a gradient is also designed on the upstream face (Fig. 164) in order to reach the required stability.

Straight masonry dams of cantilever type are suitable for channels whose slopes are not very stable. They should preferably be built onto rocky outcrops in the bottom. If the masonry is to have a broader span, the dam body is divided by vertical expansion joints (Fig. 165).

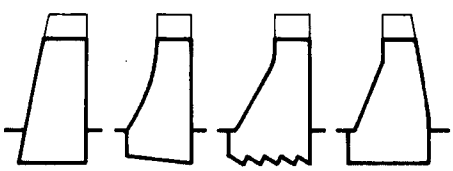


Fig.164 The shape of the body in masonry dams.

Fig.165 A masonry dam of cantilever type (Photo L. Novák).

Monolithic straight dams, not divided by vertical joints, can be built in narrow profiles having strong slopes on both sides. However, monolithic structures of subtler design have to be perfectly built and strong. The ground plan of arched dams has the shape of a sector of a circular or parabolic cylindrical surface; in some dams such a sector may form only the central part of the dam body and the wings on both sides will adjoin the centre part as tangets. The arched dam bodies are usually built in wider profiles where the use of straight dams would not be economical (use is made of both the gravity and arch effect of the dam body) and in narrow rocky profiles having a high bearing capacity (using the arch effect of the dam).

The most common shape of the spillway of stonework masonry dams is trapezoidal but it may also be dish-shaped or rectangular.

Concrete dams (Fig. 166) are built either from plain concrete on reinforced concrete. Dams of plain concrete may be either straight or arched in ground plan.

Straight concrete dams are usually designed as of gravity type. As the downstream side of such dams has a slope of 5:1 or less, it is exposed to abrasion by the suspended sediment carried by the water falling over the crest and also to the danger of frost damage. To protect the concrete of the dam, it is useful to face the dam body with a layer of quarry stones laid in cement mortar. As to arch dams (Fig. 167), the dam body is a part of a circular cylinder whose centre line is most frequently a circular arc, defined by radius and the central angle. The central line depends, in particular, on the morphological and geological conditions of the site. The most important morphological feature is the contour pattern. The ideal location of an arch dam in the terrain would be a place in which the centre line of the arch would cross the contour lines at a right angle, the smallest admissible angle being 45°.

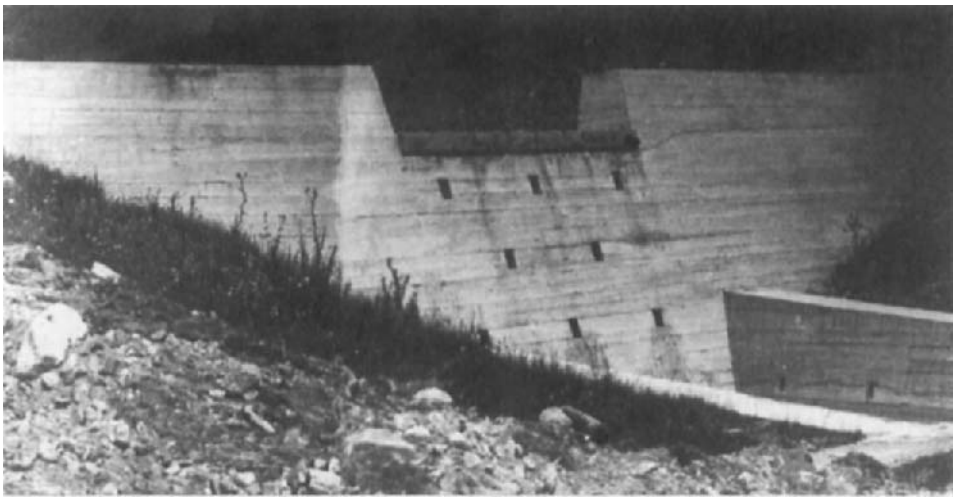


Fig.166 A concrete dam (Photo L. Novák).

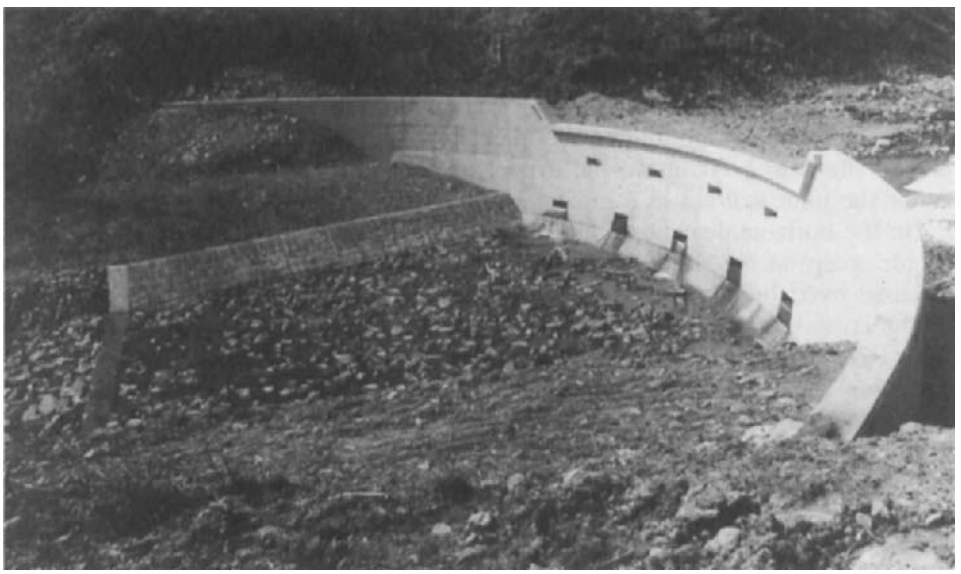


Fig.167 An arch dam on the Lužec in the Ore Mts., Bohemia (Photo Z. Macoun).

The stability of the arch depends on the geological structure of the ground. The worse the quality of the ground, the greater the danger of the abutments' yielding along the sliding surface that may have developed under them. In arches with a large central angle, the sliding surface will soon turn towards the ground surface and the passive action of the rock cover will be small. In arches with a small central angle, the sliding surface will be longer and the ground would have a higher resistance to act against the displacement of the soil under the abutments of the arch (Fig. 168).

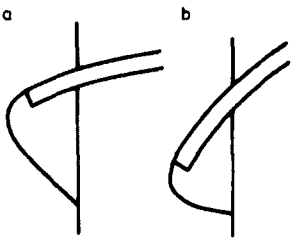


Fig.168 The sliding surface under the arch abutment.

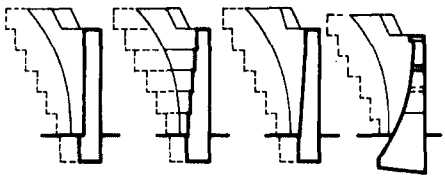


Fig.169 Cross-sections through the body of arch dams.

Hence, if the abutments of arch dams are to be safer, it is not recommended that the dam body's centre lined should have a large central angle. However, neither is too small a central angle recommended because it reduces the effect of the arch and gives rise to unfavourable tensile stresses on the upstream side of the abutment. To ensure sufficient safety of arch dams, even on unfavourable ground materials, with the dam body remaining as thin as possible, it is recommended to use a central angle of 60 – 120°.

If the central angle in the middle part of the body is smaller than 60°, the arch effect of the dam is very low in that middle part and has to be replaced by the force of gravity. In such cases, the dam body is divided by a horizontal expansion joint which will break the monolithic action of the structure in the vertical direction. The part of the dam body above the expansion joint will act as an arch and the part under the joint will act as a gravity body.

On the horizontal sections, the thickness of the arch is the same over its whole length except at the abutments where it widens to provide a better distribution of the load over the abutments.

The cross-section of an arch dam will show a rectangular or trapezoidal shape (Fig. 169). The width of a rectangular cross section may either be the same in all horizontal sections or may increase from the crest towards the foundations. In such a case, the upstream face is vertical and the downstream side retreats from the toe to the crest.

With a trapezoidal cross-section, the upstream side is vertical and the gradient of the downstream side has a slope of 10 : 1 (it may also decrease from crest to toe).

The holes in the body of the arch dam disturb the effect of the arch, so their number should be as low as possible. Their design dimensions are 0.3/0.3 m to 0.2/0.6 m. If the holes have to be larger they should be round, as this shape does not interfere with the action of the arch.

Dams built from prefabricated reinforced-concrete units. Prefabricated reinforced-concrete units are used for the construction of both gravity and arch dams. As to the gravity dams (Fig. 170), the prefabricated units are used to build "crib" type structures, the inner space being filled with rock or gravel as described in the chapter concerning weirs. It is a disadvantage of this type of structure that it combines highly productive methods of assembly with laborious manual filling. It is therefore better to use the prefabricated-parts system in arched dams where the whole dam body is assembled only from prefabricated concrete

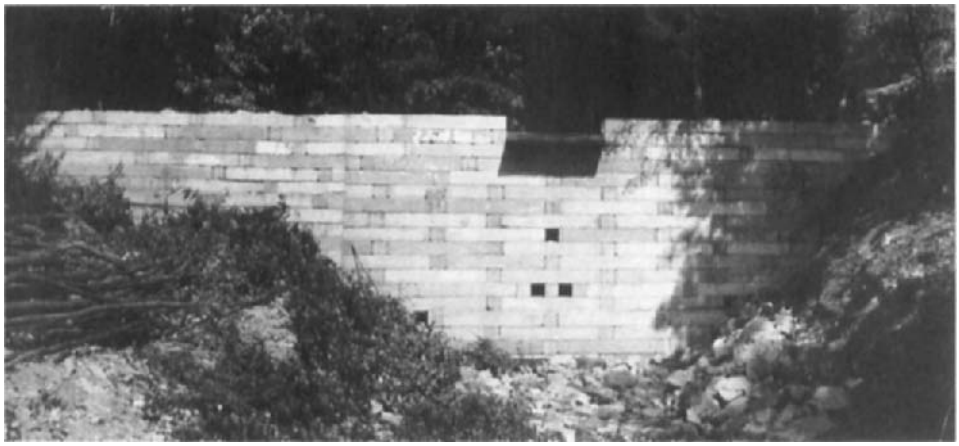


Fig.170 Gravity dam of prefabricated reinforced blocks (Photo L. Novák).

parts (Fig. 171). However, industrially produced parts cannot be used in such structures.

Wooden dams. Log dams are constructed in the same way as the log structures built to protect channel slopes. They are especially suitable for channel segments with nonstable slopes, prone to sliding. Owing to the logs' high tensile bending strength, log dams are more resistant than masonry structures to the lateral pressure developing during sliding processes.

Stone-filled gabion dams. The body of a stone-filled gabion dam (Fig. 172) is of the same structure as the stone-filled gabion walls built to protect channel slopes. On the upstream side, the body of such a dam is vertical and on the downstream side it is either vertical or retreating. The spillway may be rectangular, retreating or trapezoidal.

The wire baskets suffer from abrasion by the material carried by the water. On the spillway they are protected by round billets, stone lining, prefabricated slabs or iron sheeting.

It may sometimes also be necessary to protect the gabions on the upper surfaces of the retreating structural gravel gabion layers.

The main advantages of using gravel gabions to build dams include low cost, the use of local sources of rock and gravel and their natural appearance. Such dams are flexible and as such they remain stable although the channel slopes may be of poor stability.

Recent developments in dam construction. New approaches to the construction of dams have been developed in recent years as a result of efforts to replace materials that are in short supply (stone), to make maximum use of machines and prefab assembly systems, to reduce the need for skilled labour, to save investment costs and to reduce the time of construction. The new approaches include the use of steel elements either to build the whole body of the dam (Fig. 173) or only its middle part (Fig. 174). The dam wall may be either porous or fully closed (having only the drain holes).

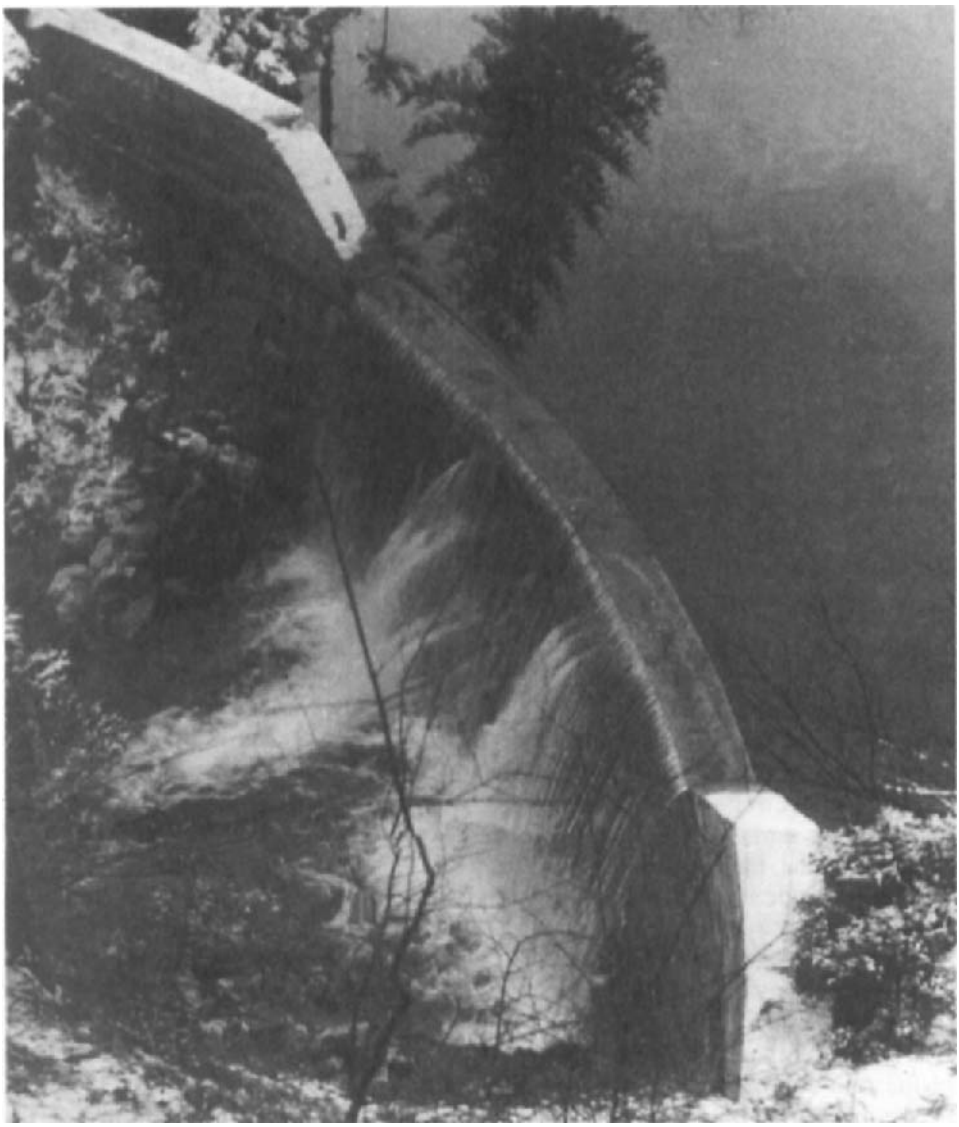


Fig.171 An arch dam of prefabricated concrete blocks on the Vadičovka torrent (Photo L. Novák).

The use of steel elements in the construction of a dam body is shown in Fig. 175.

Static calculations of the dams

The static calculations vary with the types of construction and include the selection of the basic form and dimensions of the dam body and its evaluation.

A dam body is analysed as being exposed to soil pressure, hydrostatic pressure, and a combination of hydrostatic pressure and the pressure of submerged sediments.

Fig.172 Stone-filled gabion dam (Photo K. Dimitrovský).

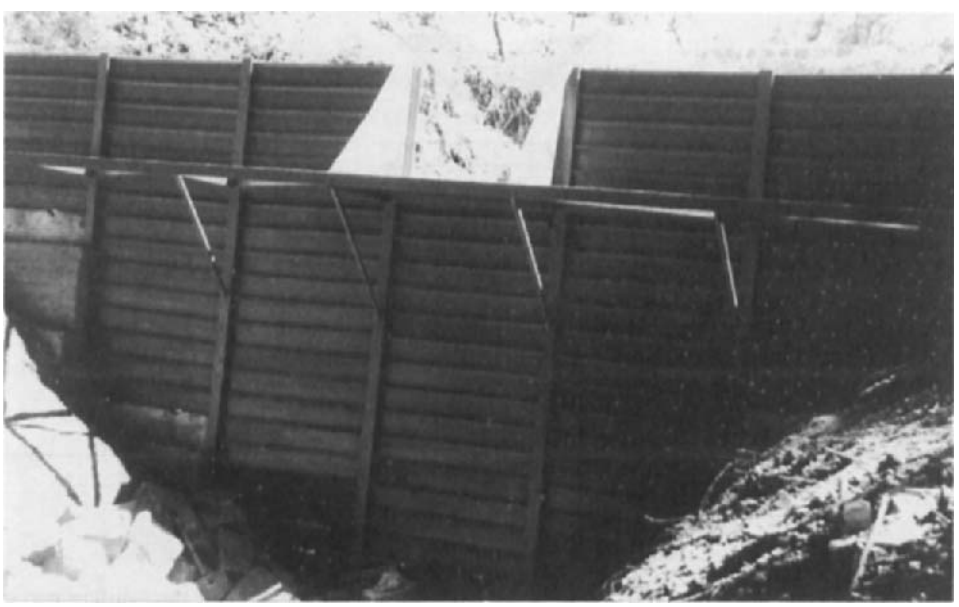


Fig.173 A steel dam (Photo L. Novák).

Fig.174 A steel dam with through-flow part (Photo L. Novák).

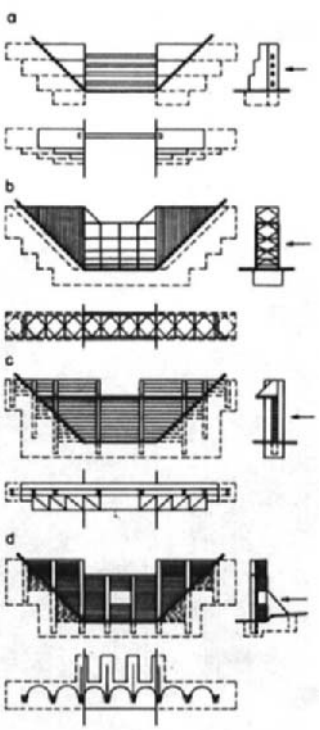


Fig.175 Examples of using steel elements in the construction of dams: a — the through-flow part of I irons, b — framed structure, c — solid-wall structure of I irons, d — solid-wall structure of "Tubosiders".

Soil pressure is only considered in those dams whose catchment basin is to be immediately filled with soil (in gullies, in torrent control projects for residential areas, etc.).

Exposure of a dam to hydrostatic pressure throughout its whole height is the most frequent type of load acting. The dams are analysed for the compressive load of the hydrostatic pressure which occurs during the 50-year flood (50 years is the assumed service life of dams and the required stability thereof should be maintained for that period). Only in the cases in which an important structure, facility, etc. on the torrent downstream from the dam is to be protected should the 100-year flood exposure also be analysed.

Analysis of simultaneous exposure to hydrostatic pressure and the pressure of submerged sediment is only considered in cases of permanent exposure to this combination of pressures. In the majority of dams on torrents, the catchment basin is successively filled with sediment of different texture. At low flow rates the gaps between the larger particles are progressively filled with finer earth material and the permeability of the sediment declines. The least favourable situation would occur if the catchment basin is immediately filled with sediment. However, as a rule this happens when the maximum flow is over, so that the structure is exposed to a much lower hydrostatic pressure.

This is also the reason why hydraulic uplift pressure is not taken into account in the static calculations.

Static calculations for cantilever-type gravity dams

In the case of cantilever-type gravity dams, a static calculation is performed for the tallest cantilever, exposed to the greatest load (Fig. 176). (Such a cantilever, 1.0 m wide, is hypothetically taken out of the body of the dam). The stability of that cantilever is analysed with respect to the possibility of its being disturbed. The following cases may occur in dams on torrents: the cantilever may turn over its downstream toe, the cantilever may be pushed, the masonry may be disturbed by tensile stress on the upstream face, or the ground, having a poor bearing capacity, may yield under the cantilever's foundations.

Overturning of cantilevers. This may happen as a result of the action of the sum of the moments of horizontal forces acting on the cantilever body.

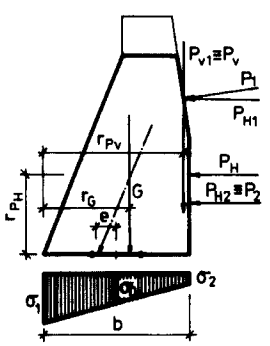


Fig.176 Indication for stress analysis of a cantilever dam.

Moments of the vertical components act against these forces. The dam's stability is broken as a result of overturning if the ratio of the sum of vertical-force moments to the sum of the moments of horizontal forces is smaller than 1. This is determined by the coefficient of security against overturning:

$$k_1 \frac{Gr_G + P_v r_{P_v}}{P_H r_{P_H}} = 1, \tag{6.148}$$

where G – weight of the cantilever ($N \cdot m^{-1}$),
r_G – cantilever gravity arm (m),
P_v – vertical component of the hydrostatic pressure ($N \cdot m^{-1}$),
r_{P_v} – arm of the vertical component of hydrostatic pressure (m),
P_H – horizontal component of the hydrostatic pressure ($N \cdot m^{-1}$),
r_{P_H} – arm of the horizontal component of hydrostatic pressure (m).

The coefficient of security against overturning $k_1 = 1$ corresponds to the extreme state. For masonry structures the coefficient of security should be $k_1 = 1.5$.

Sliding of cantilevers. A cantilever may slide if the external horizontal forces acting on the upstream side are greater than the friction acting on the foundation. The coefficient of security against sliding is determined by the relationship

$$k_2 = \frac{(G + P_v)f}{P_H}, \tag{6.149}$$

where G – weight of the cantilever ($N \cdot m^{-1}$),
P_v – vertical component of hydrostatic pressure ($N \cdot m^{-1}$),
f – coefficient of friction,
P_H – horizontal component of hydrostatic pressure ($N \cdot m^{-1}$).

The approximate values of the friction coefficient for the most common cases are shown in Tab. 71. For better security it is again necessary to raise the coefficient of security against sliding to $k_2 = 1.2$.

Table 71

Values of the friction coefficient

Type of friction	Coefficient of friction f	Angle of friction
Concrete over concrete	0.75	36°52′
Concrete over rock	0.75	36°52′
Concrete over medium-quality rock	0.50–0.65	26°34′–33°01′
Concrete over poor-quality rock	0.30–0.50	16°42′–26°34′
Concrete over sand or gravel	0.35–0.50	19°17′–26°34′
Concrete over clay	0.20–0.30	11°18′–16°42′
Stone masonry over stone masonry	0.60–0.70	30°58′–35°
Stone masonry over sand and gravel	0.50–0.60	26°34′–30°58′
Stone masonry over loamy earth	0.40–0.55	21°50′–28°50′
Stone masonry over clayey earth	0.30–0.80	16°42′–26°34′

Sliding stability is usually of key importance for the stability of the cantilever as a whole. Designers often fail to attain the required coefficient of security without unnecessarily enlarging the volume of the dam body. In such cases, the needed stability can be achieved by adjusting the gradient of the foundation surface to obtain a normal component which would form angle α with the resultant of external and internal forces. The required magnitude of the angle will be calculated by the formula

$$k_2 = \frac{f}{\operatorname{tg} \alpha}. \tag{6.150}$$

Stress in the dam body and in the bottom of the footing. The key requirements for the stability of gravity dams of cantilever type include the elimination of tensile stress on the upstream side in any joint in the dam body. If the dam body's tensile strength is exceeded, a horizontal crack occurs on the upstream side which reduces the cantilever's stability in overturning.

Tensile stress will occur on the upstream side of the footing bottom if the resultant of the external and internal forces "passes outside" the internal core third. The distance of the vertical component of the resultant from the downstream toe is given by

$$r = r_G - r_v \geq b/3 \qquad \text{(m)}, \tag{6.151}$$

where r_v – distance of the resultant's vertical component from force G (m):

$$r_v = \frac{P_H}{G + P_v} r_{p_H} \qquad \text{(m)}. \tag{6.152}$$

Stress in the footing bottom will be calculated from the formula

$$\sigma_{1,2} = \sigma_o \left(1 \pm \frac{6e}{b}\right) \qquad \text{(Pa)}, \tag{6.153}$$

where $\sigma_{1,2}$ – stress in the downstream (σ_1) and upstream (σ_2) side of the footing bottom (Pa),
σ_o – mean stress $[\sigma_o = N/(b \cdot 1.0)]$ (Pa)
N – vertical component of the resultant of external and internal forces (N),
b – width of the footing bottom (m),
e – excentricity [the distance of the vertical component from the centre of the footing bottom (m)].

If the resultant "passes outside" the internal core third it is possible, in less important dams, to solve the problem of stress with the elimination of tension:

$$\sigma = \frac{2N}{3c'} \qquad \text{(Pa)}, \tag{6.154}$$

where c' – distance of the normal component from the downstream toe. The condition is that $c' > b/6$.

Table 72

Derived standard stress q_0 (MPa) of compacted gravelly earths for a foundation depth of 1 m

Class	Gravelly earth (over 50% grains larger than 2 mm)	Material to fill gaps between large grains	Width of foundations B (m)			
			0.5	1	3	6
8	Large grains in contact with each other	Sandy earth, compacted	0.490	0.785	0.981	0.785
9		cohesive	0.294	0.490	0.686	0.490
10	Large grains not in contact with each other	Sandy earth, compacted	0.392	0.588	0.785	0.686
11		cohesive	Consistency of earth filling			
			soft	solid	strong	hard
			0.078	0.147	0.245	0.392

As a rule, the factors evaluated include the joint at the level of the lining of the downstream floor and also the footing bottom. The stress that has arisen should not exceed the allowable compressive stress of the masonry or the allowable stress acting on the supporting ground, determined either on the basis of geological survey or according to the standard stress data in Tab. 72–75.

Example 8:

The dam in Fig. 177 is designed as a gravity structure of cement stonework masonry. Its unit weight $\gamma_m = 24.525\ \mathrm{kN \cdot m^{-3}}$. The chosen dimensions of the cantilever include: width at crest $k = 1.2$ m, width in foundations $b = 2.8$ m. The supporting ground is gravelly earth, class 10.

Cantilever weight:

$$G = (k + b)0.5H \cdot 1.0\,\gamma_m = (1.2 + 2.8)0.5 \cdot 4.0 \cdot 1.0 \cdot 24.525 = 196.2\ \mathrm{kN}.$$

Hydrostatic pressure

$$P = [h + (H + h)]0.5H \cdot 1.0\,\gamma = [1.0 + (4.0 + 1.0)]0.5 \cdot 4.0 \cdot 1.0 \cdot 9.81 = 117.72\ \mathrm{kN}.$$

Overturning stability

$$k_1 = \frac{Gr_G}{Pr_P} \geq 1.5$$

r_G of Fig. 177 is $r_G = 1.7$ m

r_P of Fig. 177 is $r_P = 1.6$ m

$$k_1 = \frac{196.2 \cdot 1.7}{117.72 \cdot 1.6} = 1.77 > 1.5.$$

Stability against sliding

$$k_2 = \frac{G}{P} f > 1.2$$

Table 73

Derived standard stress q_0 (MPa) of compacted sandy earths for a foundation depth of 1 m

Class	Sandy earth (over 50% of grains smaller than 2 mm)	Granularity	Earth	Foundation width B (m)			
				0.5	1	3	6
12	coarse sand (more than 50% of grains larger than 0.5 mm)	nondispersive $d_{60}/d_{10} < 5$	dry to wet	0.294	0.490	0.785	0.558
13		monodispersive $d_{60}/d_{10} < 5$	dry to wet	0.245	0.343	0.588	0.490
14		with admixture of grains < 0.05 mm above 5%	–	0.196	0.294	0.490	0.392
		non-monodispersive and monodispersive	very wet				
			water-saturated				
15	Medium-grained sand (more than 50% grains larger than 0.25 mm)	with admixture of grains < 0.05 mm below 2%	dry to wet	0.245	0.343	0.539	0.392
			very wet	0.196	0.294	0.392	0.294
16	Fine-grained sand (more than 75% of grains larger than 0.1 mm)		water-saturated				
17	silt-like, i.e., dusty sand (less than 75% of grains larger than 0.1 mm, if $I_p > 1$)	with admixture of grains of 0.05 mm above 2%	dry to wet	0.147	0.196	0.245	0.196
18			very wet	0.098	0.147	0.196	0.147
			water-saturated				

f has a value taken from Tab. 71 $f = 0.6$

$$k_2 = \frac{196.2}{117.72} \cdot 0.6 = 1.0 < 1.2.$$

Because $k_2 < 1.2$ it is necessary either to increase the weight of the cantilever through a greater thickness of the dam body at the crest and in the foundation or to adjust the gradient of the latter so that the component of the resultant perpendicular to the footing bottom could be at an angle α' with the resultant:

$$\operatorname{tg} \alpha' = \frac{\operatorname{tg} \alpha}{k_2},$$

Table 74

Derived standard stress q_0 (MPa) of consolidated and semiconsolidated rocks for a foundation depth of 1 m

Origin of rock			Min. strength (MPa)	Amount of cracks		
				low	medium	high
Class	igneous or metamorphous	sedimentary		disintegration produces small parallelepipeds with edge lengths (in cm): over 25	6 to 25	below 6
Mineral rocks						
1	sound	sound, sheet-jointed (layers over 25 cm)	58.840	5.844	3.923	1.961
2	partially weathered (part of minerals weathered, sound minerals in contact with each other)	partially weathered sheet-jointed, sound tabular (layers 5 to 25 cm)	14.710	1.961	0.981	0.588
3	weathered (prevailing part of minerals weathered)	weathered sheet-jointed, partially weathered tabular, sound thin-layered (layers thinner than 5 cm)	not prescribed	0.981	0.588	0.392
4	disintegrated (evaluated according to table)	weathered tabular, partially weathered thin-layered	not prescribed	–	0.392	0.294
Semimineral rocks (e.g., claystones, slates)						
5	sound, consolidated		9.807 – 1.961	0.392 – 0.588		
6	partially weathered, medium consolidated		1.961 – 0.981	0.294 – 0.392		
7	weathered, slightly consolidated		0.981 – 0.294	0.196 – 0.294		

$\alpha = 22°07' \ldots \operatorname{tg} \alpha = 0.40626,$

$$\operatorname{tg} \alpha' = \frac{0.40626}{1.2} = 0.33855 \ldots \alpha = 18°42'.$$

Table 75

Derived standard stress q_v (MPa) for cohesive earths for a foundation depth of 0.8 to 1.5 m

Class	Consistency of the earth (plasticity index $I_p > 1$)	Consistency			
		soft	solid	strong	hard
19	with a low plasticity $I_p < 10(7)$ (silt, sandy silt, clayey silt)	0.078	0.147	0.245	0.392
20	with medium plasticity $10(7) < I_p$ 20(17) (sandy clay, silt clay)	0.069	0.118	0.245	0.392
21	with a high plasticity $I_p > 20(17)$ (clay)	0.049	0.098	0.196	0.343
22	dry loess (grains of 0.01 to 0.05 mm)	0.147–0.196			

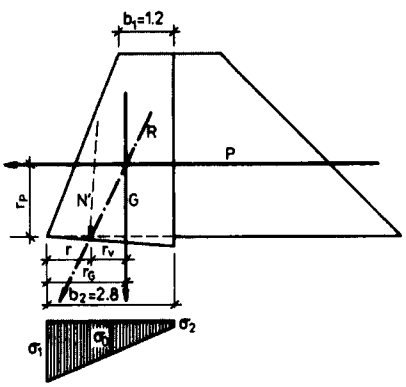


Fig.177 Indication for stress analysis of the cantilevers in Example 8.

It is proposed to adjust the gradient of the foundation.

Stress in the foundation

Determining the position of the resultant:

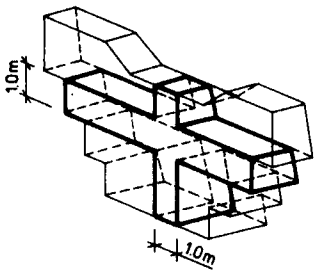
$$r = r_G - r_v$$

$$r_v = \frac{P}{G} r_P = \frac{117.72}{196.2} 1.6 = 0.96 \text{ m}$$

$r = 1.7 - 0.96 = 0.74 < (b/3) = 0.93 \ldots$ the resultant passes outside the middle third and the stress therein is evaluated with the elimination of tension ($c' = r = 0.74 > 0.47 = b/6$):

$$\sigma = \frac{2N}{3c'} = \frac{2 \cdot 196.2}{3 \cdot 0.74} = 176.76 \text{ kPa} < 708.17 \text{ kPa} = q_o$$

(standard stress q_o for earth of class 10 is taken from the data in Tab. 72; for foundation width $3c' = 2.22$ m).



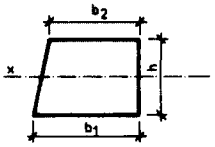


Fig.178 The load-bearing elements of the body of a monolithic dam.

Fig.179 A cross-section through the horizontal beam of a monolithic dam.

Static calculations of monolithic dams

If the dam body is to act as a monolithic, it should be free of vertical joints and should be perfectly built so as to have sufficient tensile strength against bending.

KRONFELLNER – KRAUS (1962) found that the tensile strength against bending, measured in the horizontal elements of the masonry, reaches values of 1.06–3.28 MPa. Taking into account the lower value with a 2.5-fold security, the admissible bending tensile stress for cement stonework masonry will be 0.4 MPa.

Being keyed into the banks, monolithic dams can have a three-dimensional action. In the static calculation it is assumed that the dam body consists of horizontal strips [stretching over the whole length of the dam (Fig. 178) and acting as beam] and of vertical cantilevers.

The horizontal beams take over part of the total load and transmit it into the banks. The remaining load is transmitted by gravity effect into the foundations.

A monolithic dam is, in essence, a plate of uneven thickness, supported on two sides (supporting on the third side is not considered because if the foundations are undercut this third support would be lost).

The procedure of stress analysis is as follows:

1. Divide the designed dam body into hypothetical horizontal beams, usually 1.0 m high, and select a tallest vertical cantilever (in the middle of the dam) whose width would be 1.0 m.

2. Use the equation

$$p_n = \frac{8M}{l^2} \qquad (\mathrm{N \cdot m^{-1}}), \tag{6.155}$$

where p_n – part of water pressure transmitted through the beam ($\mathrm{N \cdot m^{-2}}$),
M – maximum bending moment of the beam ($\mathrm{N \cdot m}$),
l – the span of the beam (m),
to determine the part of water pressure transmitted by the beam (per 1 m of the beam's length).

Load the beams to their maximum bearing capacity, i.e., expose them to the maximum permissible stress. Determine the maximum bending moment the rise of which is still admissible, using the following formula:

$$M = W_x \sigma_{\mathrm{allow}} \qquad (\mathrm{N \cdot m}), \tag{6.156}$$

where W_x – section modulus of the beam's cross-section (m^3),
σ_{allow} – allowable bending tensile stress of the masonry (Pa).
The section modulus is determined by the equation

$$W_x = \frac{I_x}{e_x} \quad (m^3), \tag{6.157}$$

where I_x – inertia moment of the trapezoidal cross-section of the beam (m^4),
e_x – distance between the marginal filaments of the beam (m).

The moment of inertia and the distance between the two marginal filaments (Fig. 179) are calculated from the following two formulae:

$$I_x = \frac{h}{36} \frac{b_1^4 + 2b_1b_2(b_1^2 + b_2)^2 + b_2^4}{b_1 + b_2} \quad (m^4), \tag{6.158}$$

$$e_x = \frac{3b_1^2 - (b_1 - b_2)^2}{3(b_1 + b_2)} \quad (m), \tag{6.159}$$

where h – height of cross-section (m),
b_1 – the longer of the parallel sides of the trapezoidal cross-section (m),
b_2 – the shorter of the parallel sides of the trapezoidal cross-section (m).

The values of I_x, e_x and W_x for the most frequently occurring cross-sections at $h = 1$ m are shown in Tab. 76.

3. Using the equation for total load determines the proportion transmitted by the cantilever

$$p_i = p_{n_i} + p_{k_i} \quad (N \cdot m^{-2}), \tag{6.160}$$

where p_i – total load of the part of the dam body concerned ($N \cdot m^{-2}$),
p_{n_i} – load transmitted by the beam ($N \cdot m^{-2}$),
p_{k_i} – load transmitted by the cantilever ($N \cdot m^{-2}$).

The solution should proceed from the top beam (from the crest) to the foundations.

4. With the cantilever the only condition to be satisfied is that the resultant of the external and internal forces should remain within the middle third of the footing bottom. If this were not so the amount of load taken over by the beam would by greater than the allowable load (determined by calculation). The total load to which the cantilever would be exposed is a sum of partial stresses:

$$p_k = \sum p_{k_i}, \tag{6.161}$$

This total pressure force as a resultant of all the partial loads to which the cantilever is exposed from the crest to the foundations has its point of action above the footing bottom at a height of

$$r_p = \frac{\sum(p_{k_i} r_{p_i})}{p_k} \quad (m), \tag{6.162}$$

Table 76

Values of I_x, e_x and W_x for trapezoidal profiles

b_2	b_1	I_x	e_x	W_x
0.6	0.8	0.03	0.45	0.07
0.7	0.9	0.04	0.50	0.09
0.8	1.0	0.06	0.55	0.11
0.9	1.1	0.08	0.60	0.14
1.0	1.2	0.11	0.65	0.17
1.1	1.3	0.15	0.70	0.21
1.2	1.4	0.19	0.75	0.25
1.3	1.5	0.23	0.80	0.29
1.4	1.6	0.28	0.85	0.33
1.5	1.7	0.34	0.90	0.38
1.6	1.8	0.41	0.95	0.43
1.7	1.9	0.49	1.00	0.49
1.8	2.0	0.57	1.05	0.55
1.9	2.1	0.67	1.10	0.61
2.0	2.2	0.78	1.15	0.67
2.1	2.3	0.89	1.20	0.74
2.2	2.4	1.02	1.25	0.81
2.3	2.5	1.16	1.30	0.89
2.4	2.6	1.31	1.35	0.97
2.5	2.7	1.47	1.40	1.05
2.6	2.8	1.64	1.45	1.13
2.7	2.9	1.83	1.50	1.22
2.8	3.0	2.04	1.55	1.31
2.9	3.1	2.25	1.60	1.41
3.0	3.2	2.49	1.65	1.51

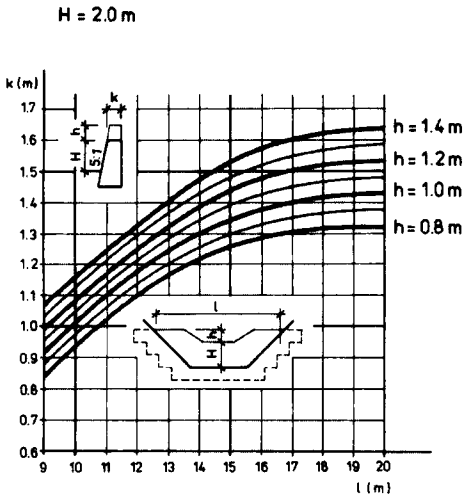
where r_{p_i} – distance between horizontally acting partial loads acting on the cantilever (m).

Calculations by equations (6.151) and (6.152) will allow one to estimate if the resultant force remains within the middle third of the foundation or not. With the cantilever it has to be considered whether the stress that develops in the foundation exceeds the allowable stress to which the latter can be exposed.

The static calculation for monolithic dams is facilitated by nomograms in Figs. 180 – 183 determining the thickness of the dam at the spillway crest in dependence on its span (NOVÁK, 1973).

Example 9

The dam in Fig. 184 has the following main dimensions: $H = 3.0$ m, $h = 1.0$ m. The designed slope of the downstream side is 5 : 1 and the width of the crest is $k = 1.4$ m. The ground material is gravelly earth of class 10.



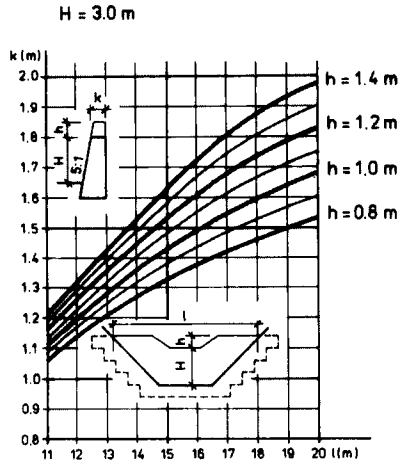
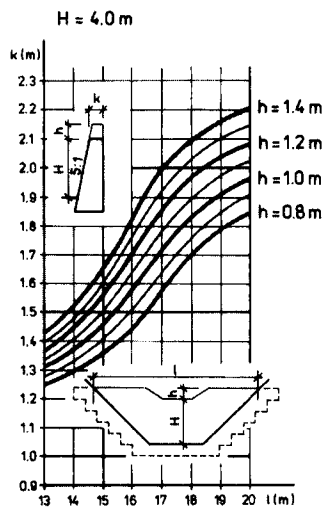


Fig.180 Nomogram for designing the crest thickness of a monolithic dam (H = 2.0 m).

Fig.181 Nomogram for designing the crest thickness of a monolithic dam (H = 3.0 m).



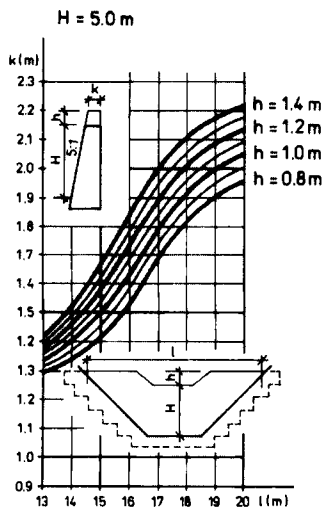


Fig.182 Nomogram for designing the crest thickness of a monolithic dam (H = 4.0 m).

Fig.183 Nomogram for designing the crest thickness of a monolithic dam (H = 5.0 m).

Down from the crest, the dam's height is divided into four horizontal strips, each 1.0 m high.

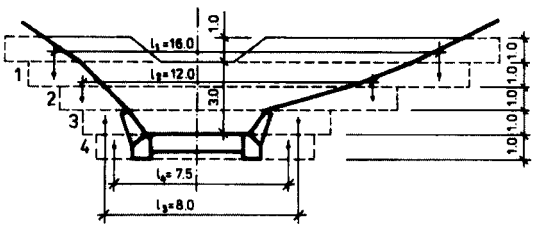
a) Distribution of load in the horizontal strips:

Strip 1:

Basic dimensions: strip height $y_1 = 1.0$ m, $b_1 = 1.4$ m.

$b_2 = 1.6$ m, $l_1 = 16.0$ m.

I_x, e_x and W_x values can be found in Tab. 76:



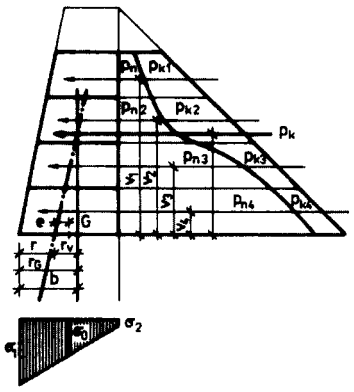


Fig.184 Indication of stress analysis for monolithic dam in Example 9.

Fig.185 Calculation of cantilever stability for the monolithic dam in Example 9.

$I_x = 0.28\ \text{m}^4$, $e_x = 0.85\ \text{m}$, $W_x = 0.33\ \text{m}^3$

$M_1 = W_x \sigma_{\text{allow}} = 0.33 \cdot 400 = 134\ \text{kNm}.$

The hydrostatic pressure in the first strip is $p_1 = \dfrac{10 + 20}{2} = 15\ \text{kN} \cdot \text{m}^{-2}.$

The load transmitted through the horizontal beam is $p_{n1} = \dfrac{8 \cdot 134}{16.0^2} = 4.19\ \text{kN} \cdot \text{m}^{-2}.$

The remaining load to be transmitted through the cantilever is $p_{k1} = 15 - 4.19 = 10.81\ \text{kN} \cdot \text{m}^{-2}$

The distance of the p_{k1} from the foundation (in Fig. 185) is $r_{p1} = 3.4\ \text{m}$

Strip 2:

$b_1 = 1.6\ \text{m}$, $b_2 = 1.8\ \text{m}$, $l_2 = 12.0\ \text{m}$, $y_2 = 1.0\ \text{m}.$

$I_x = 0.41\ \text{m}^2$ $e_x = 0.95\ \text{m}$ $W_x = 0.43\ \text{m}^3$

$M_2 = 0.43 \cdot 400 = 172\ \text{kNm}$

$p_2 = 25\ \text{kN} \cdot \text{m}^{-2}$

$p_{n2} = \dfrac{8 \cdot 172}{12^2} = 9.55\ \text{kN} \cdot \text{m}^{-2}$

$p_{k2} = 25 - 9.55 = 15.45\ \text{kN} \cdot \text{m}^{-2}$

$r_{p2} = 2.5\ \text{m}$

Strip 3:

$b_1 = 1.8\ \text{m}$, $b_2 = 2.0\ \text{m}$, $l_3 = 8.0\ \text{m}$, $y_3 = 1.0\ \text{m}$

$I_x = 0.57\ \text{m}^4$ $e_x = 1.05\ \text{m}$ $W_x = 0.55\ \text{m}^3$

$M_3 = 0.55 \cdot 400 = 220\ \text{kNm}$

$p_3 = 35\ \text{kN} \cdot \text{m}^{-2}$

$p_{n3} = \dfrac{8 \cdot 220}{8^2} = 27.50\ \text{kN} \cdot \text{m}^{-2}$

$p_{k3} = 35 - 27.50 = 7.5\ \text{kN} \cdot \text{m}^2$

$r_{p3} = 1.5\ \text{m}$

Strip 4:

$b_1 = 2.0\,\text{m}$, $b_2 = 2.2\,\text{m}$, $l_4 = 7.5\,\text{m}$, $y_4 = 1.0\,\text{m}$

$I_x = 0.78\,\text{m}^4$ $\quad e_x = 1.15\,\text{m}$ $\quad W_x = 0.67\,\text{m}^3$

$M_4 = 0.67 \cdot 400 = 268\,\text{kNm}$

$p_4 = 45\,\text{kN}\cdot\text{m}^{-2}$

$$p_{n4} = \frac{8 \cdot 268}{7.5^2} = 38.12\,\text{kN}\cdot m^{-2}$$

$p_{k4} = 45 - 38.12 = 6.88\,\text{kN}\cdot\text{m}^{-2}$

$r_{p4} = 0.5\,\text{m}$

b) Assessment of the cantilever:

The load taken over by the cantilever is

$$p_k = p_{k1} + p_{k2} + p_{k3} + p_{k4} = 10.81 + 15.45 + 7.5 + 6.88 = 40.64\,\text{kN}\cdot\text{m}^{-2}.$$

The point of action of pressure force:

$$r_p = \frac{\sum(p_{ki} r_{pi})}{p_k} = \frac{10.81 \cdot 3.4 + 15.45 \cdot 2.5 + 7.5 \cdot 1.5 + 6.88 \cdot 0.5}{40.64} = 2.22\,\text{m}.$$

The weight of the cantilever is $G = (1.4 + 2.2)0.5 \cdot 4.0 \cdot 1.0 \cdot 24.525 = 176.58\,\text{kN}$.
The area of force G from Fig. 185 is $r_G = 1.3\,\text{m}$

$$r_v = \frac{p_k}{G} r_p = \frac{40.64}{176.58} 2.22 = 0.51\,\text{m}.$$

The distance of the resultant force from the downstream edge of the foundation is

$$r = r_G - r_v = 1.3 - 0.51 = 0.79\,\text{m} > \frac{b}{3} = \frac{2.2}{3} = 0.73\,\text{m}.$$

Stress in the foundation:

$$\sigma_o = \frac{G}{b \cdot 1.0} = \frac{176.58}{2.2 \cdot 1.0} = 80.26\,\text{kPa}$$

$$\sigma_{1,2} = \sigma_o \left(1 \pm \frac{6e}{b}\right)$$

$e = 0.3\,\text{m}$

$$\sigma_1 = 80.26 \left(1 + \frac{6 \cdot 0.3}{2.2}\right) = 145.93\,\text{kPa} < 706\,\text{kPa} = q_o$$

$$\sigma_2 = 80.26 \left(1 - \frac{6 \cdot 0.3}{2.2}\right) = 14.59\,\text{kPa} < 706\,\text{kPa} = q_o$$

Static calculation of arch dams

For arch dams it is assumed that all the load to which the dam body is exposed is transmitted into the slopes of the structure profile solely through the action of the arch. The stress analysis for arch dams can be performed by the method of separate nonelastic thin-walled rings or by the more accurate method of separate elastic rings.

The elementary method of nonelastic rings determines the stress arising in the rings when their surface is exposed to load, e.g., by formula

$$\sigma = \varrho \frac{hr}{d} g \qquad \text{(Pa)}, \tag{6.163}$$

where ϱ – water density ($kg \cdot m^{-3}$),
h – water depth corresponding to the ring under study (m),
e – radius of the upstream side of the ring (m),
d – average thickness of the arch ring (m).

By the more accurate method, the rings are treated as elastic arches fixed in the footings and exposed to uniform and radial load. The stress arising in the crown and abutments of the arch is calculated by this method.

For stress analysis based on the elastic arch method, the designed body is divided into rings of a stable height, each being treated separately, irrespective of their interactions (Fig. 186).

Solutions based on this method have been described by a number of authors. The procedure developed by GUIDI can be recommended because it is very simple. Guidi's formulae are used to determine the stress at the crown and abutments of the ring: for the ring crown (upstream, downstream):

$$\sigma_{v_{1,2}} = p(\varrho_k \pm \mu_{1,2}) \qquad \text{(Pa)}, \tag{6.164}$$

for the ring abutment (upstream, downstream):

$$\sigma_{p_{1,2}} = p(\varrho_k \mp \mu'_{1,2}) \qquad \text{(Pa)}, \tag{6.165}$$

where p – water pressure exerted on the central line of the arch (Pa),
ϱ_k – the radius-thickness ratio of the arch,
$\mu_{1,2}, \mu'_{1,2}$ – coefficients expressing the effect of the shape of the arch.

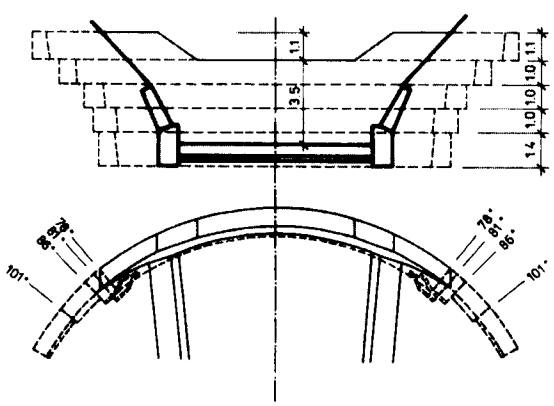


Fig.186 Division of the body of an arch dam into horizontal rings.

The hydrostatic pressure exerted on the central line of the arch is calculated from the known pressure to which the upstream side of the arch is exposed

$$p = p_o \frac{r + \frac{d}{2}}{r} \quad \text{(m)}, \tag{6.166}$$

where p_o – pressure exerted on the upstream side of the arch (Pa).

The radius-thickness ratio ϱ_k of the arch is

$$\varrho_k = \frac{r}{d}. \tag{6.167}$$

The coefficients $\mu_{1,2}$ and $\mu'_{1,2}$ are calculated from the equation

$$\mu_{1,2} = \frac{1}{D}\left(\frac{s-1}{s} \mp \frac{d}{6r}\right), \tag{6.168}$$

$$\mu'_{1,2} = \frac{1}{D}\left[\frac{1}{s} - \frac{r-f}{r}\left(1 \mp \frac{d}{6r}\right)\right], \tag{6.169}$$

$$D = \frac{s}{l} + \frac{r-f}{r} - \frac{2l}{s} + \frac{d^2}{6r^2}\left(2\frac{s}{l} - \frac{r-f}{r}\right), \tag{6.170}$$

where s – length of the arch's centre line (Fig. 187) (m),

l – length of the chord (m),

f – rise at the centre line's apex (m).

The calculation of stress can be speeded up by using the nomograms shown in Figs. 188–191 (NOVÁK, 1969).

Stress at the crown of the arch should not exceed the allowable compressive stress for concrete. The stress at arch abutments should not exceed the allowable stress to which the ground soils may be exposed. If the stress at the crown of the arch is higher than the maximum allowable stress for concrete, the shape of the arch should be changed either through increased thickness or through a larger central angle. If the stress at the crown of the arch corresponds to the maximum allowable

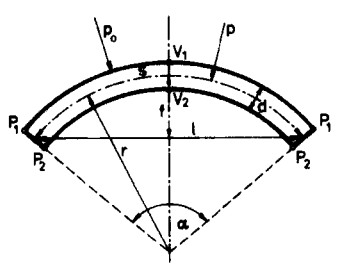


Fig.187 Basic parameters of an arch ring.

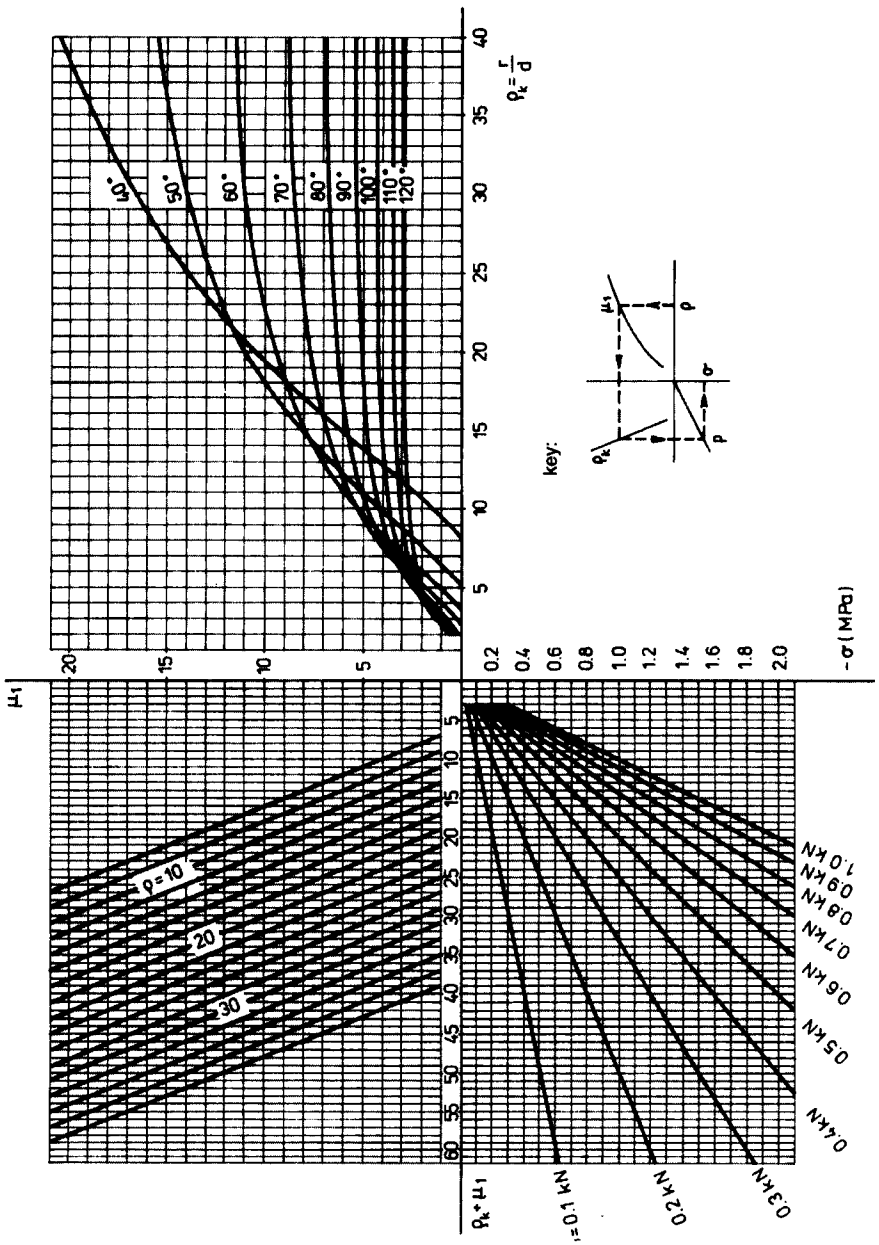


Fig.188 Nomogram for the determination of stress in the upstream arch crown.

stress for concrete and the stress at the abutment exceeds the stress allowable for the ground soil, the arch can maintain its shape and the abutment should be extended to reduce the stress to an adequate allowable stress. The extension of the abutment should be larger on its downstream side if the stress is to be, as far as

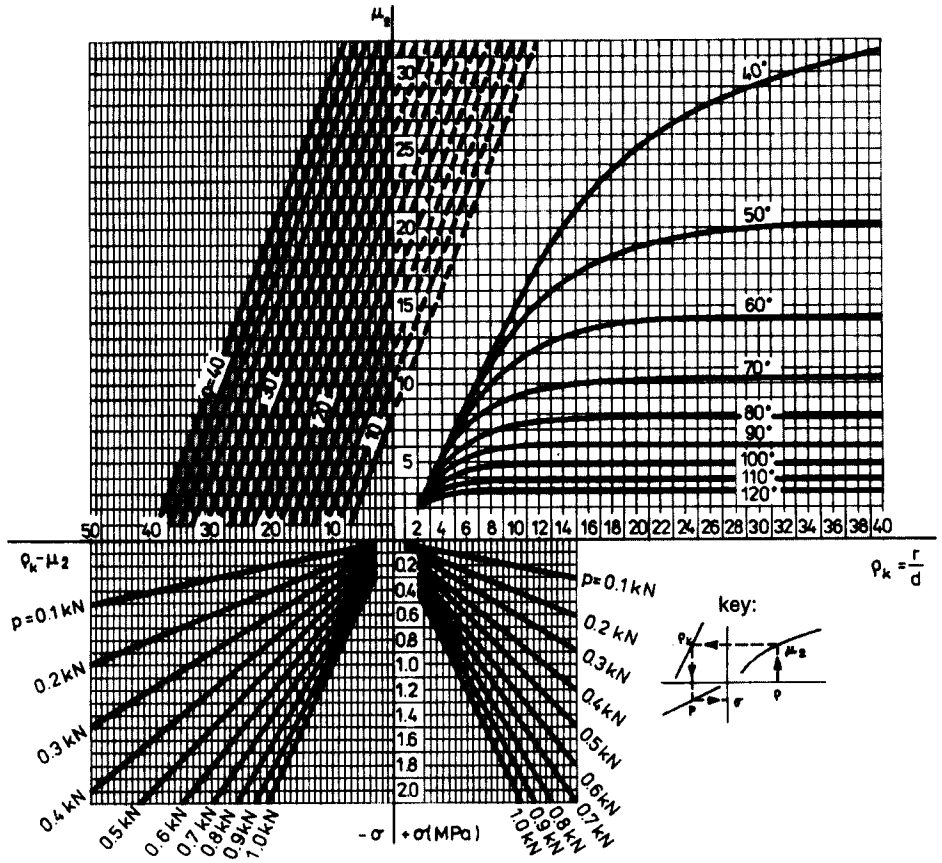


Fig.189 Nomogram for the determination of stress in the downstream arch crown.

possible, uniform throughout the abutment. The required impost area will be calculated from the formula

$$S' = \frac{S\sigma_o}{\sigma_{\text{allow}}} \quad (\text{m}^2), \tag{6.171}$$

where S – area of the original abutment of the arch (m^2),
σ_o – average stress arising in the original abutment (Pa),
σ_{allow} – allowable stress exposure of the abutment (Pa).

If the stress in the abutment is to be uniform, the resultant of the external and internal forces should pass through the centre of the extended footing bottom. According to Fig. 192, the extension of the abutment will be determined by the equation

$$a = \frac{d'}{2} + e \quad (\text{m}), \tag{6.172}$$

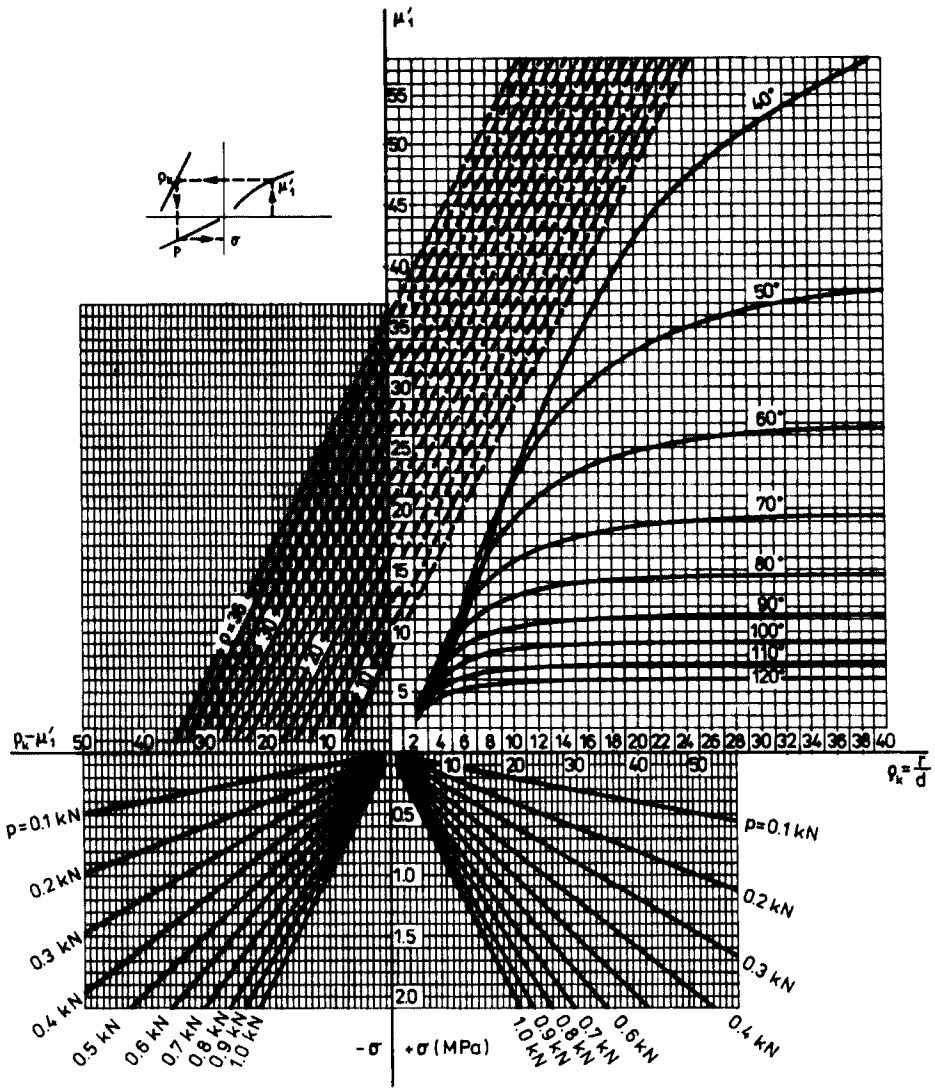


Fig.190 Nomogram for the determination of stress in the upstream arch abutment.

where d' – width of extended abutment (m),
e – excentricity of the resultant (m).

The width of the extended abutment is

$$d' = \frac{S'}{h} \quad \text{(m)}. \tag{6.173}$$

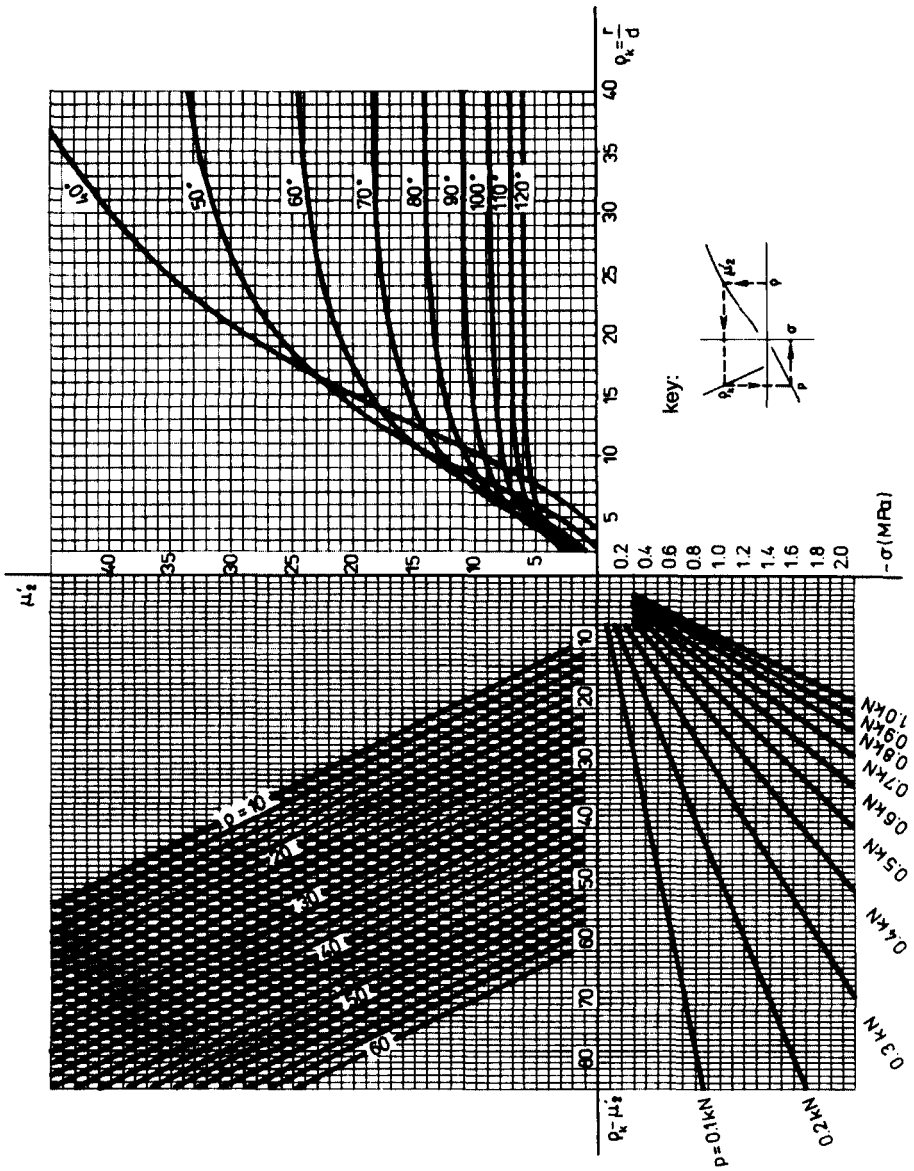


Fig.191 Nomogram for the determination of stress in the downstream arch abutment.

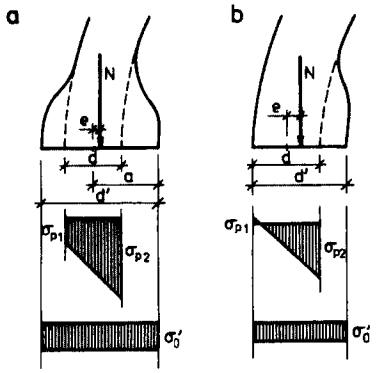


Fig.192 Extension of arch abutment: a — bilateral, b — unilateral.

Excentricity is calculated from the equation for stress

$$\sigma_{p_2} = \sigma_o \left(1 + \frac{6e}{dh}\right),$$

$$e = \frac{dh(\sigma_{p_2} - \sigma_o)}{6\sigma_o}, \tag{6.174}$$

where h — height of the ring (m).

Example 10

The dam in Fig. 186 has the following dimensions: $H = 4.6$ m, $h = 1.1$ m. The downstream side is designed to have an inclination of 10:1. The dam's thickness at crest is $d = 0.75$ m, the radius of the upstream side is $r = 11.5$ m.

The height of the dam is divided into four horizontal rings 1.0 m and 1.4 m high. The allowable stress to which the abutment may be exposed is 400 kPa and the allowable compressive stress exposure for concrete is 3430 kPa.

a) Determining the stress

First ring:
$\alpha_1 = 101°$, $d_1 = 0.8$ m, $r_1 = 11.1$ m, $h_1 = 1.0$ m.
To be able to determine the values of $\mu_{1,2}$ and $\mu'_{1,2}$ it is first necessary to calculate the radius-thickness ratio

$$\varrho_{k_1} = \frac{r_1}{d_1} = \frac{11.1}{0.8} = 13.88.$$

The values of $\mu_1 = 4.2$, $\mu_2 = 4.7$, $\mu_1' = 8.8$, $\mu_2' = 8.2$ are determined from Fig. 188 – 191 for ϱ and α.

The pressure exerted on the upstream side of the ring is

$$p_{o1} = \frac{11 + 21}{2} = 16 \text{ kPa}.$$

The pressure exerted on the central line of the arch is

$$p_1 = p_{o1} \frac{r_1 + \frac{d_1}{2}}{r_1} = 16 \frac{11.1 + 0.8 \cdot 0.5}{11.1} = 16.60 \text{ kPa}.$$

Stress at arch crown

$$\sigma_{v_1} = p_1(\varrho_{k_1} + \mu_1) = 16.60(13.88 + 4.2) = 300.13 \text{ kPa},$$

$$\sigma_{v_2} = p_1(\varrho_{k_1} + \mu_2) = 16.60(13.88 - 4.7) = 152.39 \text{ kPa}.$$

Stress at arch abutment

$$\sigma_{p_1} = p_1(\varrho_{k_1} + \mu_1') = 16.60(13.88 - 8.8) = 84.33 \text{ kPa},$$

$$\sigma_{p_1} = p_1(\varrho_{k_1} + \mu_2') = 16.60(13.88 + 8.2) = 366.53 \text{ kPa}.$$

Because the stress at both the head of the arch and at the abutment is below the allowable stress level, the designed shape and dimensions of the ring will be satisfactory.

Second ring:

$\alpha = 86^\circ \quad \delta_2 = 0.9 \text{ m} \quad r_2 = 11.05 \text{ m} \quad h_2 = 1.0 \text{ m}$

$$\varrho_{k_2} = \frac{11.05}{0.9} = 12.26 \text{ kPa},$$

$\mu_1 = 5.0 \quad \mu_2 = 6.6 \quad \mu_1' = 11.9 \quad \mu_2' = 10.8$

$p_{o_2} = 26 \text{ kPa},$

$$p_2 = 26 \frac{11.05 + 0.45}{11.05} = 27.06 \text{ kPa}$$

$\sigma_{v_1} = 27.06(12.28 + 5.0) = 467.60 \text{ kPa} < 3430 \text{ kPa}$

$\sigma_{v_2} = 27.06(12.28 - 6.6) = 153.70 \text{ kPa}$

$\sigma_{p_1} = 27.06(12.28 - 11.9) = 10.28 \text{ kPa}$

$\sigma_{p_2} = 27.06(12.28 + 10.8) = 624.54 \text{ kPa} > 400 \text{ kPa}$

Because $\sigma_{p_2} > \sigma_{\text{allow}}$, it is necessary to extend the abutment of the arch:

$$e = \frac{\sigma_{p_2} - \sigma_o}{6\sigma_o} dh,$$

$$\sigma_o = \frac{\sigma_{p_1} + \sigma_{p_2}}{2} = \frac{10.28 + 624.54}{2} = 317.41 \text{ kPa},$$

$$e = \frac{654.54 - 317.41}{6 \cdot 317.41} 0.9 \cdot 1.0 = 0.15 \text{ m},$$

$$d' = d + 2e = 0.9 + 2 \cdot 0.15 = 1.2 \text{ m}.$$

The extended surface of the abutment is

$S' = d'h = 1.2 \cdot 1.0 = 1.2 \text{ m}^2.$

The uniform stress in the extended abutment is

$$\sigma_o' = \frac{\sigma_o d}{d'} = \frac{317.41 \cdot 0.9}{1.2} = 238.06 \text{ kPa} < 400 \text{ kPa}$$

Third ring:

$\alpha_3 = 81°$ $d_3 = 1.0$ m $r_3 = 11.0$ m $b_3 = 1.0$ m

$$\varrho_{k_3} = \frac{11.0}{1.0} = 11.0$$

$\mu_1 = 5.0$ $\mu_2 = 7.3$ $\mu_1' = 13.0$ $\mu_2' = 11.0$

$p_{o_3} = 36$ kPa

$$p_3 = 36\,\frac{11 + 0.5}{11} = 37.64 \text{ kPa}$$

$\sigma_{v_1} = 37.64(11 + 5.0) = 604.24$ kPa < 3430 kPa

$\sigma_{v_2} = 37.64(11.0 - 7.3) = 139.27$ kPa

$\sigma_{p_1} = 37.64(11.0 - 13.0) = -75.28$ kPa

$\sigma_{p_2} = 37.64(11.0 + 11.0) = 828.08$ kPa > 400 kPa

The tensile stress that arises on the upstream side of the abutment is within the allowable limits. Extension of the abutment:

$$\sigma_o = \frac{828.08 - 75.28}{2} = 376.40 \text{ kPa},$$

$$e = \frac{828.08 - 376.40}{6 \cdot 376.40}\,1.0 \cdot 1.0 = 0.2 \text{ m},$$

$$d' = 1.0 + 2 \cdot 0.2 = 1.4 \text{ m},$$

$$S' = 1.4 \cdot 1.0 = 1.4 \text{ m}^2,$$

$$\sigma_o' = \frac{376.40 \cdot 1.0}{1.4} = 268.86 \text{ kPa} < 400 \text{ kPa}.$$

Fourth ring:

$\alpha_4 = 78°$ $d_4 = 1.1$ m $r_4 = 10.54$ m $h_4 = 1.4$ m,

$$\varrho_{k_4} = \frac{10.54}{1.1} = 9.33,$$

$\mu_1 = 4.8$ $\mu_2 = 7.7$ $\mu_1' = 13.2$ $\mu_2' = 10.8$,

$p_{o_4} = 48$ kPa,

$$p_4 = 48\,\frac{10.54 + 0.55}{10.54} = 50.50 \text{ kPa},$$

$\sigma_{v_1} = 50.5(9.33 + 4.8) = 713.56$ kPa < 3430 kPa,

$\sigma_{v_2} = 50.5(9.33 - 7.7) = 82.32$ kPa,

$\sigma_{p_1} = 50.5(9.33 - 13.2) = -195.44$,

$\sigma_{p_2} = 50.5(9.33 + 10.8) = 1016.56$ kPa $>$ kPa.

Extension of the abutment:

$$\sigma_o = \frac{1016.56 - 195.44}{2} = 410.56 \text{ kPa},$$

$$e = \frac{1016.56 - 410.56}{6 \cdot 410.56} 1.1 = 0.27 \text{ m},$$

$$d' = 1.1 + 2 \cdot 0.27 = 1.64 \text{ m},$$

$$s' = 1.64 \cdot 1.4 = 2.30 \text{ m}^2.$$

$$\sigma_o = \frac{410.56 \cdot 1.1}{2.3} = 196.35 \text{ kPa} < 400 \text{ kPa}.$$

CHAPTER 7

Gully Control

7.1 Gully erosion

Gully erosion is the the most destructive kind of erosion; it reduces the area of land available for farming. Its destructive manifestations include scouring of the land, deposition of sediment on growing crops, worsening of soil quality in the accumulation zone, general drying of the affected regions by interrupting the groundwater table, silting of river channels and pollution of surface waters.

Though most of the erosion damage caused to farmland is ascribed to sheet erosion, the losses due to gully erosion are also considerable and what is worse, they are irreversible. The majority of the gullies developed back in the past when extensive farming prevailed. At present, the manifestations of gully erosion in European countries are not so severe and wide-spread as they used to be; nevertheless, much damage is still caused to farmland even today owing to inexpert site planning and inadequate farming practices. Much of the gully erosion takes the form of renewed erosion activity in apparently stabilised gullies.

7.2 Causes of gully erosion

In Central Europe, the rise and development of gullies as the most dangerous stage of accelerated water erosion is usually due to a disturbance of the equilibrium of natural conditions, caused by poor soil management. The intensity of gully processes depends on the terrain's proneness to erosion, i.e., on the erosibility of the soil and the parent rock underneath, on the nature and distribution of precipitation, on the geomorphology of the area and on the vegetation cover.

Gully erosion is a consequence of an increased scouring activity of concentrated surface runoff which may develop on very wide slopes as a result of the concentration of areal runoff in rills and furrows or in concave depressions in inadequately aligned roads, road drain ditches, field boundaries across the contour lines, wheel tracks and furrows. Gullying in man-made surface runoff collectors is, in fact, much more dangerous than the formation of gullies as a higher stage of water erosion on arable land.

The proneness of certain areas to gullying can be evaluated by analysing the natural factors and factors associated with human activities, and the interactions of both. The latter factors should be modified to fit into the natural conditions to prevent gully processes (by controlling the causes) even in areas very susceptible to gullying.

7.2.1 Climatic conditions

The areas of the greatest susceptibility to gullying are, first of all, inland regions with intermittent flood rains and long dry periods. Such areas include, for example, the Ukraine, the vast plains along the Russian river Volga, the prairie regions of the USA and others. The factor most important for gully development is not the total annual rainfall but the occurrence of short flood rains during which the soil is not able to absorb a substantial part of the precipitation water. In the areas prone to gully erosion, such rains most frequently occur in spring and summer when the soil is either oversaturated from the snowmelt or cracked and nonporous after a long drought.

7.2.2 Geomorphology

The geomorphological factors that influence the occurrence of gullies and development of gully processes include the topography, the steepness of slopes and their width, shape and exposure. According to SPIRHANZEL (1952), the critical slope gradient for scour and gully formation is 8% (5°). MAŘAN (1953) believes that erosion furrows may develop on arable land at slope gradients higher than 8 – 10°, i.e., 14.1 – 17.6°. These data are suggested just for general orientation and are far from generally applicable. Development of gully erosion depends on a number of factors, the key role always being played by the factor prevailing under the given conditions (e.g., a high erosibility of the soil).

7.2.3 The soil and geological conditions

Gully processes are influenced by the underlying ground layer which acts both as parent rock controlling the nature of the soil on the slope and as subsoil which itself may be affected by the deep scours and gullies. The substrates most susceptible to erosion include the soft rocks of the flysh zone, the soft and crumbly rocks of volcanic origin (tuffs, agglomerates), Permian and calcareous sandstones and sand-clay sediments in Tertiary basins.

7.2.4 Vegetation

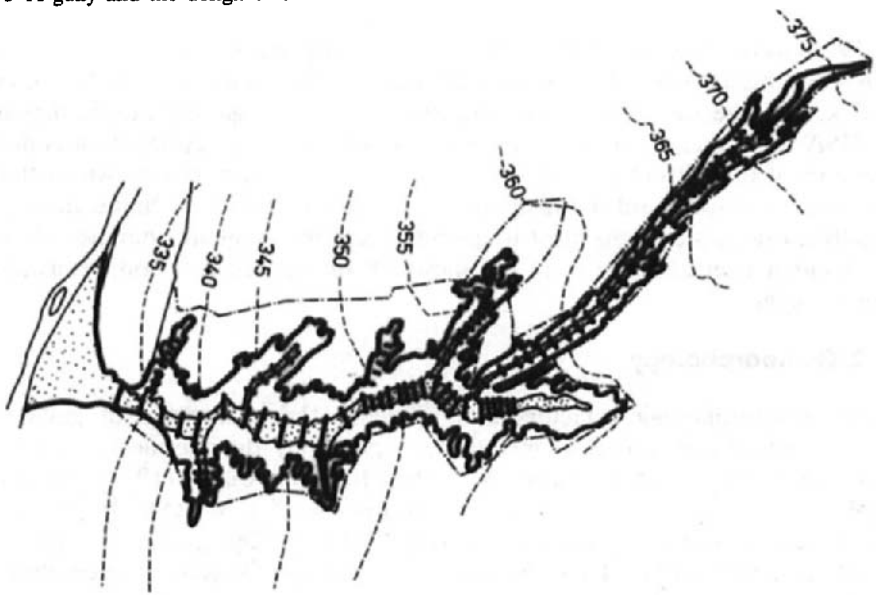
Vegetation cover reduces the danger of gullying: plants reduce the force of falling raindrops, maintain a favourable soil structure by controlling evaporation, and act favourably on the soil's infiltration capacity.

The greatest importance is attached to vegetation above the gully head. The plants' root systems stabilise the soil surface and reduce surface runoff: it is necessary that the crops grown above the gully head have a high potential for the infiltration of water into the soil.

7.3 Gully forms

Gullies may develop either from erosion furrows or as separate formations in terrain depressions, hollowed-out farm tracks, etc. (Fig. 193).

Fig.193 A gully and the design of control measures.



The gully begins with a headcut (Fig. 194) in the soil which advances by headward erosion into the headwater area. The bottom of the gully head has a

Fig.194 Headcut of the gully (Photo D. Zachar).

Fig.195 The channel of the gully (Photo D. Zachar).

steep gradient. Depending on local conditions, the gully may branch into multiple extensions.

The gully head passes into the gully channel (Fig. 195) in which the gradient of the bottom is lower and the bank slopes are steep. As the gully bottom deepens and the slopes are undercut, the slopes lose stability. They slip down into the water which carries the earth away and the bottom of the gully widens. In this stage the process of gully channel formation depends mainly on the geological substrate.

If hard rocks lie close to the surface of the terrain, the process of bottom deepening stops at their level and the gully begins to gain width rather than depth.

A dejection cone may develop at the gully's mouth where coarser sediment is left by the water stream as it abruptly loses some of its tractive force. Finer soil particles are deposited farther from the gully mouth.

7.4 Gully stabilisation

Gully erosion passes through several stages of development. The stages of furrow formation and deepening (and widening) are the most important from the viewpoint of erosion control. It is in these stages that measures should be taken to stop further growth of the gully and to reclaim the whole area damaged by erosion.

The first thing to do is to remove the causes of erosion and to control surface runoff to prevent it from damaging the soil. During the initial stage of gully

development (while the gully is still just a deep furrow which may still be reclaimed by filling up, ploughing and dragging), gully control only requires a change in the management of the affected land, using adequate cultural practices.

However, if gully development has reached a more advanced stage, the gully process cannot be checked by interventions within its watershed but vegetation and structural measures have to be applied inside the gully itself.

7.4.1 Headcut control

Headward advance of the gully's nickpoint — hence the growth of the gully in length — can be prevented by reducing the amount of surface water getting into the gully and by lining the headcut.

Interception ditches (Fig. 196) which conduct surface runoff away from the endangered gully head will serve for completely stopping the inlet of surface runoff to the gully. An interception ditch should have its mouth in a shallow branch gully and the mouth should have the form of a lined channel or a rough-surfaced chute. However, if this method is used, the land is divided and its cultivation becomes more complicated. It is therefore better to maintain the gully's original inlet for the surface runoff and to take measures in the headwater area to slow down surface runoff and encourage the surface water to infiltrate into the soil.

This can be achieved by installing lath fences in the gully headcut and to establish a forest stand with a dense undergrowth of shrubs in the headwater area.

The establishment of a brush head plug will sometimes suffice to stabilise the headcut if it is shallow and narrow (Fig. 197).

This is done by laying conifer brushwood (pine is the best source for this purpose) onto the headcut bottom, starting at the highest point and proceeding downstream towards the gully channel itself. The thick ends of the brushwood are stuck into the bottom, the thin end being left to point against the slope of the headcut. Poles nailed to stakes will fix the layer of brushwood to the ground and stones are put on top to provide additional weight. The small soil particles carried

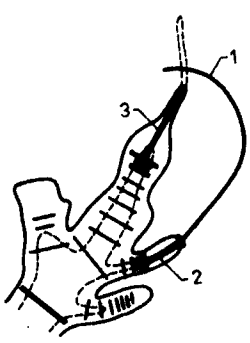
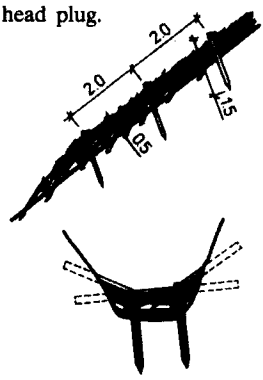


Fig.196 Gully stabilisation: 1 — interceptional ditch, 2 — lined channel of a gully branch, 3 — stabilisation of the gully head by a brush head plug in the main gully arm.

Fig.197 Brush head plug.



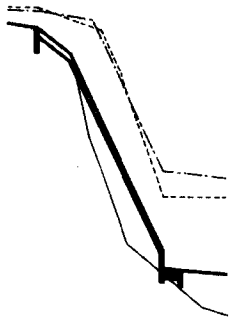


Fig.198 Lined gully head channel with a dam at the end.

by the water are entrapped in the brushwood which helps the layer to become more compact. This method of lining widens the bottom of the headcut and reduces the height of the water column.

Where the headcut is wide and deep, the best protection is to gradually raise the bottom of the gully and more or less level off the terrain. To achieve this, taller dams are combined with transverse lath fences. Only in very deep headcuts is it useful to build a rough-surfaced chute or a paved channel with a dam at the downstream end (Fig. 198).

7.4.2 Protecting the gully bottom

Shallow gullies in sandy earths, where the slopes are not steep and the bottom is wide and has a low gradient, can be progressively stabilised only by installing so-called vegetation filters, i.e., belts of woody species that can tolerate being embedded in sediment (earth) such as willows and poplars. These belts intercept the material carried by water and prevent it from being carried downstream into a river or lake or deposited on cropland. This method of gully stabilisation proceeds from the downstream reaches of the gully upstream towards the head-water area, so the upstream segments of the gully are left unprotected for some time — hence, it can only be used in extensively managed regions. In areas damaged by gullying which are to be used for intensive farming, it is necessary immediately to stop the erosive action of water entirely. In such cases a one-off technical and biotechnical solution is needed.

The purpose of the technical and biotechnical measures is to create conditions under which surface water can be safely conveyed along the gully. In uncontrolled gullies the velocity of the water is higher than the earth in which the gully is cut can bear; this velocity must be reduced through an increased resistance of the channel and through a reduction of the channel-bed slope. The resistance of the bottom and side slopes of the gully channel can be increased by grassing and afforesting the whole cross section: tree and (especially) shrub stands will mechanically fix the soil and prevent water from carrying it away; at the same time, their aerial parts roughen the surface of the flow profile and reduce the velocity of the water running along the gully channel. It often happens that gully afforestation alone brings about an equilibrium between the resistance of the bottom and the tractive force of the water. However, in the majority of cases afforestation has to be combined with an adequate reduction of the gully channel bed slope, thus preventing further deepening of the gully.

While adjusting the channel-bed gradient, the gully slopes are levelled to a lower gradient, the earth material thus obtained being put onto the gully bottom which in this way is progressively widened. The wider the bottom the shallower the water stream running along the gully channel during flood rains – and the shallower the stream the lower its scouring capacity. Considering this, the channel-bed slope in the gully can be adjusted to a higher degree than the gradients currently used in gully control. Of course, the stable bed slope has to be carefully calculated beforehead.

The bed slope can be reduced by installing simple structures for which locally available materials can be used. The intervention usually includes a combination of taller dams, which stabilise the earth fill on the bottom, with smaller secondary structures regulating the bottom between the tall dams to an adequate stable slope (Fig. 199). Almost all types of dams normally built in torrent control are suitable for gully control. Special attention has to be paid to gullies cut in loess where only masonry dams can be built – such dams should have no drain holes and should be thoroughly keyed into the slopes.

The low transverse structures that can be used in gully control include wooden sills, low transverse stone packings, earth dams lined by sodding or dry paving on the upstream side, etc. In gullies in loessic earths, even these small structures must be nonporous (e.g., protected by a clay curtain).

7.4.3 Securing the gully side slopes

Levelling of the gully side slopes (Fig. 200) is another technical intervention in the gully morphology. This operation is performed simultaneously with the regulation of the gradient of the gully channel bed. The safest gradient to which the gully side slope is to be adjusted is what is called the natural gradient of the earth. However, in deeper gullies (usually those cut in clayey to clay-sand loams) the adjustment of the slope to a natural gradient (for example, the natural gradient for clays in a gully 6 to 9 m deep is 1 : 2.75) would require the extension of the gully to a considerable width which would take up too much land. Nevertheless, the side slopes must be immediately afforested so that the protective action of the woody species can be partly relied on: the trees' roots will penetrate deep into the soil and protect it against slippage. In such cases, the gully slopes can be adjusted to a higher gradient (1 : 1–1 : 1.5) in dependence on gully depth.

The final stage of gully control is grassing or afforestation, which depends on the size of the gully and on the intended management of the reclaimed area. Woody species typical of the location are used for gully afforestation. The most suitable conifers are pines (Scotch and Austrian pine) and the best deciduous species include alders (*Alnus glutinosa* and *A. incana*), linden, maple, oak, beech, hornbeam and others. Trees will take some time before they start serving their role of soil protection, so it is advisable to provide temporary protection of the gully bottom by grassing or by planting willow cuttings.

Woody species should be planted as soon as the gully side slopes are adjusted; their root systems will contain the shearing forces that develop in the earth on the side slope and protect the slope surface against erosion. Deep-rooting species, producing an amply branched root system with a large cross-sectional area, such

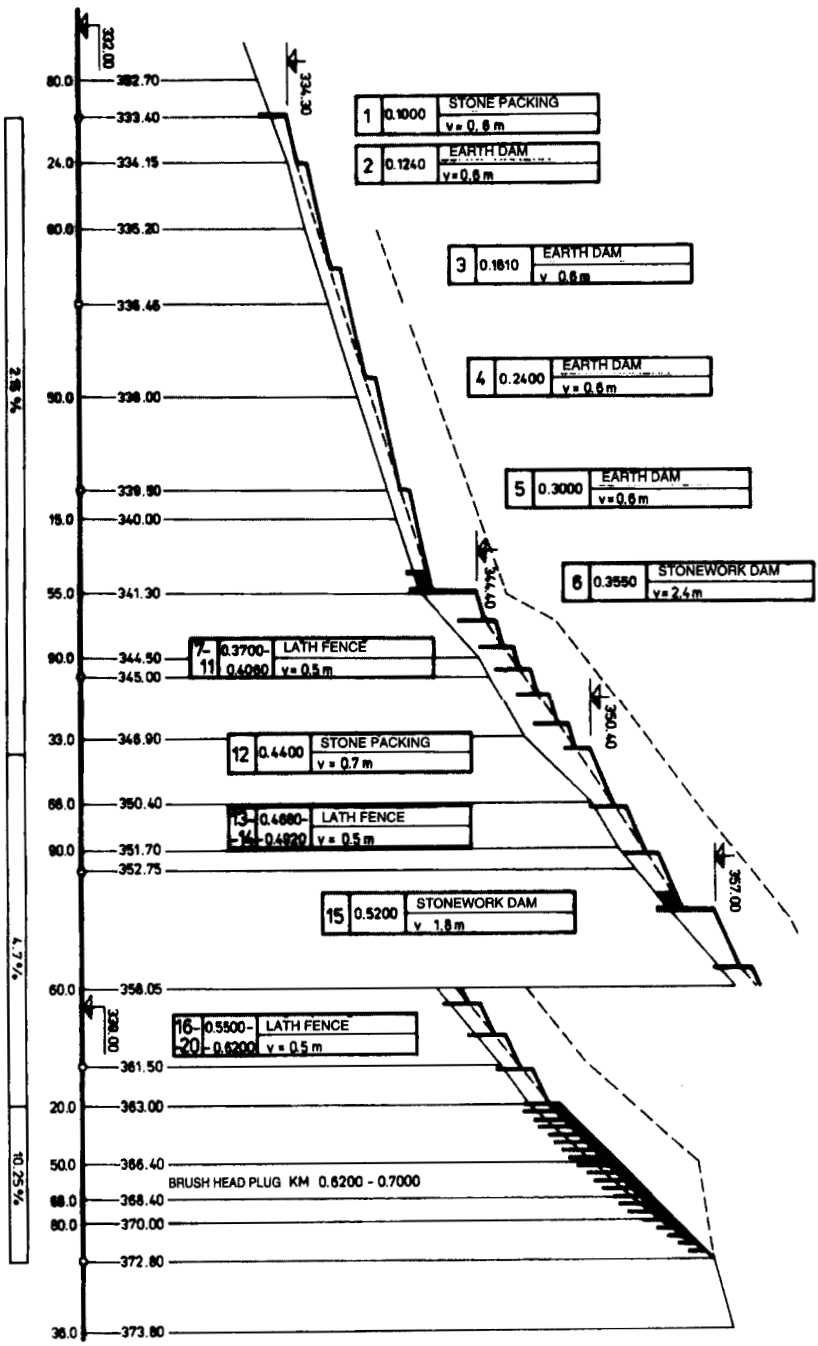


Fig.199 Adjustment of the bottom slope of a gully.

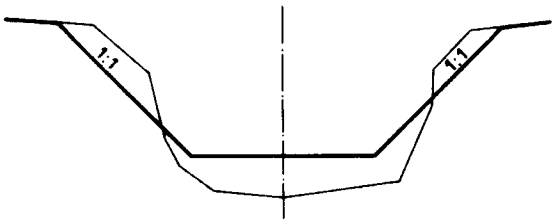


Fig.200 Adjustment of the profile of a gully channel.

as oak, maple, alder and pine, are planted at the toe of the slope. Species with a strong root system but a modest aerial part should be planted in the upper part of the side slope. Shrubs may be used for this purpose in extreme cases where the slopes are very high and the earth is unstable.

Special attention should be paid to the undergrowth which has to cover the soil surface perfectly and fix the upper layers of the soil with their roots, thus preventing them from being washed down; the roots of the undergrowth plants and shrubs will also prevent the worsening of some soil properties (soil cracking softening, disturbance of compactness, reduction of internal coherence and the angle of internal friction). In this way the undergrowth contributes to the over-all stability of the side slope.

Gullies are afforested only on the gully area and, to a necessary extent, in the adjacent headwater area.

The described method of gully control is suitable for gullies that developed in agricultural land damaged by erosion which, however, is to be further used for farming purposes.

7.5 Stabilisation of aged gully formations

Gullies that developed in uncultivated idle land or aged gully formations, usually with a permanent water flow, are controlled by methods similar to those used in torrent management. The headcut of such gullies is protected in the same way as in gullies in farmed areas.

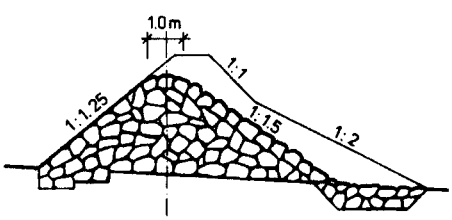


Fig.201 The body of a porous check dam.

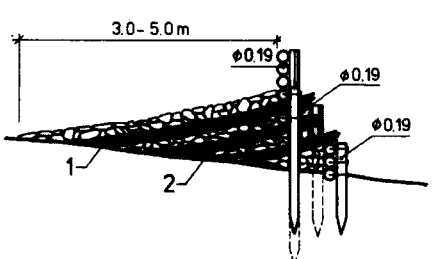


Fig.202 The body of a brushwood dam:
1 — brushwood, 2 — gravel.

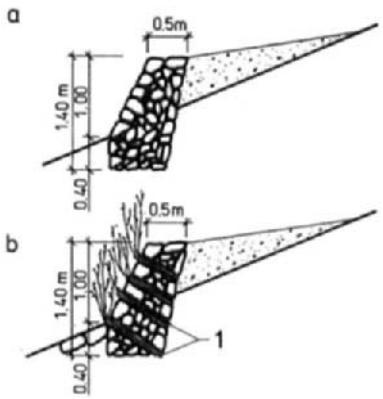


Fig.203 Stone packing: a — nonvegetated, b — vegetated, 1 — live willow brushwood.

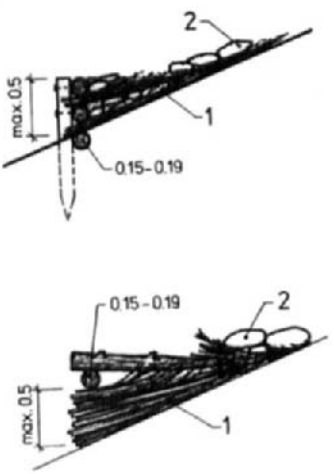


Fig.204 Brushwood sills: 1 — brushwood, 2 — stones.

The stability of the gully channel bed is provided by a system of transverse structures with a catchment basin in which the material carried by water from the upstream gully segments is deposited. The bottom is raised and widened during this process. The amount of sediment to be intercepted is the main factor influencing the decision as to which type of transverse structure to use. In addition to the common types used for torrent control, two other dam types are suitable for gully control: porous check dams and check dams of brushwood.

Porous check dams have a core of stone packing, a lining, and a surface in which the stones form a pavement-like structure. The dam is fixed by a buttress on the downstream side (Fig. 201). Water flows through the body of the porous dam and the fine soil particles suspended in the water are entrapped in its core. Thus, the whole dam body progressively becomes impermeable. As the gaps between the stones are filled, the dam gains strength and stability. When the catchment basin is filled, the water runs over the spillway and chute.

Check dams of brushwood (Fig. 202) have several rows of log walls fixed to piles. The thick ends of the brushwood are stuck into the gaps between the logs. Gravel is spread onto each layer of brushwood and the uppermost layer is loaded with stones. Conifer tops can also be used instead the brushwood.

Taller dams are built farther downstream where the gully channel is wider. Low structures built from locally available material (stone packing, brushwood sills) are installed in the upper gully segments and in the erosion furrows cut in the gully side slopes.

Material for the stone packing to check the furrows is usually found in the vicinity of the site. The stones should be arranged into a trapezoidal profile (Fig. 203). The space behind the structure is either left to be filled by a natural process or is filled with the earth from the levelled furrow slopes. In locations where bushy willow species occur, it is recommended to vegetate the stone packing using fresh

willow brushwood: once they take root, they will produce thick transverse growths shading the ground and trapping the soil particles suspended in water.

Brushwood sills (Fig. 204) are installed at sites where plenty of wood is available. They consist of an about 0.5 m layer of the brushwood of different woody species laid onto the bottom of the scour and fixed with round wood laid across and nailed to stakes driven into the bottom or embedded into the sides of the furrow. The brushwood loaded with stones can be replaced by tree tops. It is also recommended to use vegetated sills of fresh willow brushwood.

In aged gully formations where erosion processes have started anew, water may scour the bottom and undercut the side slopes. Because no material is transported to these gullies from the headwater area, it is of no avail to build a system of consolidation dams though, in fact, such interventions are often resorted to. In such gullies, it is necessary to prevent the deepening of the bottom through a system of low sills, brushwood dams and weirs with temporary catchment basins (Fig. 205) or by lining the high-gradient segments of the channel with a paving of dry stones laid in a wooden grillage or with rock chutes. Taller structures, usually weirs, are installed in places where gully processes cut deep scours in the bottom with a vertical upstream wall. The undercut side slopes of these gullies have to be secured by building retaining walls, e.g., walls of dry stone masonry, log walls, gabion walls, etc.

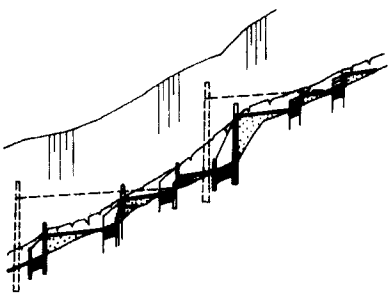


Fig.205 Adjustment of the bottom slope of a gully formation.

In both gully types, the side slopes' original gradient is maintained: they are only afforested to gain stability and resistance to erosion. If the slopes are already overgrown with woody species, the gaps in the stand are filled in with newly planted trees and trees of shallow-rooting species are removed, thin stands are underplanted, and shrub undergowth is added. Because these slopes are usually affected by local landslides, the soil must have been consolidated beforehand by technical or biotechnical methods (described in the chapter on soil protection afforestation) before afforestation starts.

CHAPTER 8

Management of Lands Endangered by Slippage

The surface of the Earth has been forming during its entire development. In periods of geological folding and discharges of lava from volcanoes, the natural stability of whole territories was broken. Such disturbances were conducive to tension in the rocks. Gravity forces, together with the internal friction forces they induced inside rocks, triggered the displacement of rock masses, i.e., landslides. At present, the equilibrium and stability of territories is most frequently disturbed by human activity in the landscape. The whole surface of the Earth is, in fact, endangered by land slippage. The death toll and economic losses caused by landslides are considerable every year. Landslides may destroy individual houses or sweep away whole settlements, devastate farm and forest land, destroy mines, cut railways, roads, bridges, tunnels, cables, surface mains and pipelines, and damage dams, channels, protecting walls, etc. Such losses may even be caused by comparatively small movements (in the order of millimetres) of the Earth's surface. The masses of slipping earth or rock may also cause enormous secondary losses; they may dam whole valleys and create large temporary or permanent water reservoirs.

Landslides have tragic consequences in densely populated mountainous areas. According to HEIM (1932), up to that time more the 5000 people had been killed and enormous economic losses caused by landslides in Switzerland. In 1806, the Tertiary conglomerates deposited on the southern slopes of the Rosberg hill moved more than 2 km and buried the town of Goldau, killing 457 people. These examples illustrate the seriousness of the problems of land slippage and show how important it is to solve them.

8.1 Causes of slope failure

Many factors, which often interact, can trigger landslides. ZÁRUBA and MENCL (1969) divide the factors responsible for land slippage as follows:

Changes in slope gradient. The causes of such changes may be both natural and induced by human activity. They include tectonic processes (subsidence or rise of the terrain), removal of the toe of the slope by water running in the channel of a watercourse (transverse erosion), or mining at the slope toe. Such changes cause changes in the tension inside the surface layers of earth or rock: if the equilibrium of these forces is broken, the slope will fail.

Excess load of the slope surface. Dumping large amounts of earth or rubble breaks the equilibrium of the forces in the ground. If a dump is located in the upper part of a slope, the tension in the ground changes and increases and if the critical tension is exceeded, the slope starts moving. Excess load may also cause an increase in pore pressure which reduces the stability of the inclined

terrain. The more rapid the growth of the dump, the greater the danger of landslide. During rapid dumping, the effect of the weight of the material may combine with the momentum of the material being unloaded from the truck.

Shocks and vibrations. These phenomena occur mainly during earthquakes and/or may be due to big explosions. Vibrations are commonly produced by machines working in the terrain. The decisive factor is represented by the different frequences of vibrations (elicited in the rocks) which are conducive to changes in tension and in the state of equilibrium in the sloping terrain.

Changes in water content in rocks. These changes are among the most important effects exerted on bottom stability. They may occur in the following forms:

Precipitation. Rain and snow water infiltrates into the soil, giving rise to hydrostatic pressure and increasing the pore pressure in the ground which, in turn, is conductive to changes in the consistence of the earth and to a decline in cohesion and internal friction in the earth material (Figs. 206, 207, 208).

The changes in electric potential on the boundaries of the different horizons cause an increase in the supply of water to the sliding surface, thus creating conditions for slippage. Different electric potentials rise at the place of contact of different materials having different electric properties.

Drying up of clayey earths. During such processes cracks occur in clayey earths and the material reduces its volume. When water is again supplied, the cracks may convey it down to the underlying rocks and as the clay imbibes moisture, the state of equilibrium is disturbed in the clay itself and in its interface with the underlying rock.

Fluctuation of ground water table. This factor is especially important when the water level declines rapidly in canals and water reservoirs. With a fall of water level in a stream, the depression curve follows such momentary changes: a high gradient of subsurface water occurs in this way. The velocity of water flow in the earth reaches a critical value at which fine soil particles are washed out of the soil profile, the earth loses its stability and the slope fails.

Effect of groundwater. These effects vary but all can disrupt the stability of the territory. Groundwater flow gives rise to hydrodynamic pressure on the soil particles which affects the stability of the slope.

Leaching of binding material from the earth. This factor reduces cohesion at the points of contact of earth particles, thus leading to a reduction of viscosity and the coefficient of internal friction.

Occurrence of subsurface hollows. If fine particles are leached out as water flows inside the slope, the soil surface sinks and internal friction inside the earth declines.

A tense water table exerts upward pressure on the underlying strata and reduces the internal friction coefficient.

Effects of frost. Water freezing in earth causes changes in the earth's volume. This enlarges cracks and reduces the strength of earths and soils. Small layers of ice develop in clays and sandy loams and as the ice thaws, the water content in the surface layers increases, sliding surfaces develop and landslides may occur.

Weathering of rocks. Mechanical and chemical weathering reduces the strength of rocks. As water infiltrates into clayey earths, changes such as hydration and ion exchange occur in such earths, also creating conditions for landslip.

Changes in vegetation cover. The trees that grow on slopes reinforce the soil with their roots and regulate water content in the surface layers of the soil. If the trees are removed, changes occur in the water regime of the surface layers which may lead to extensive landslides, erosion and complete destruction of cultivated land.

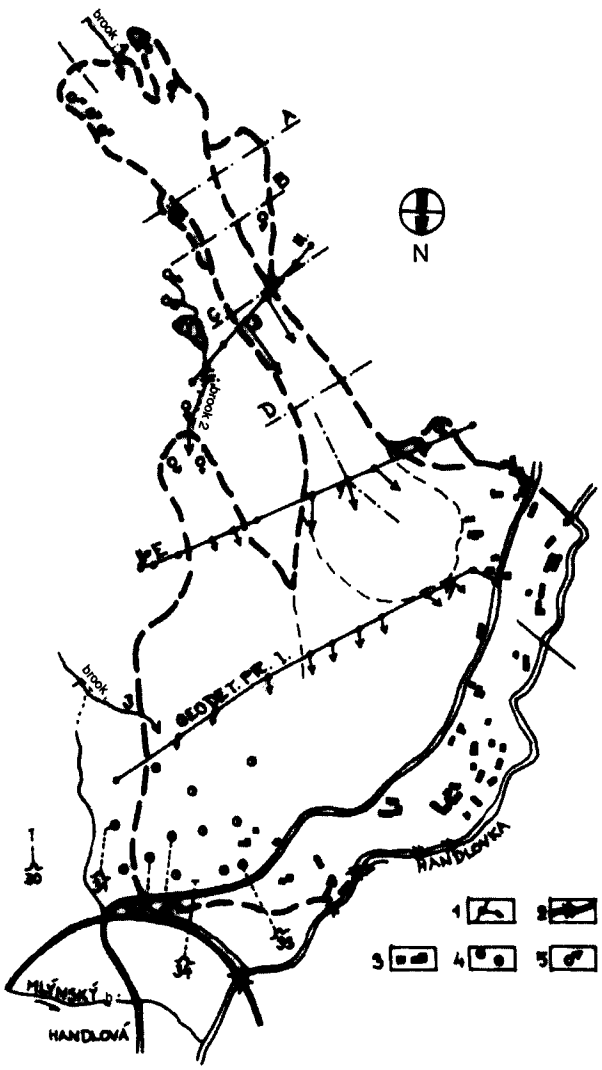


Fig.206 Landslip near Handlová, Slovakia, in 1960–1961, site plan: 1 – direction of the movement of the earth mass from January to May 1961, 2 – bottom barriers on the Handlovka brook, 3 – damaged and destroyed buildings, 4 – hydrogeological boreholes with submersion pumps, 5 – strong water springs in the landslide front (after ZÁRUBA and MENCL, 1969).

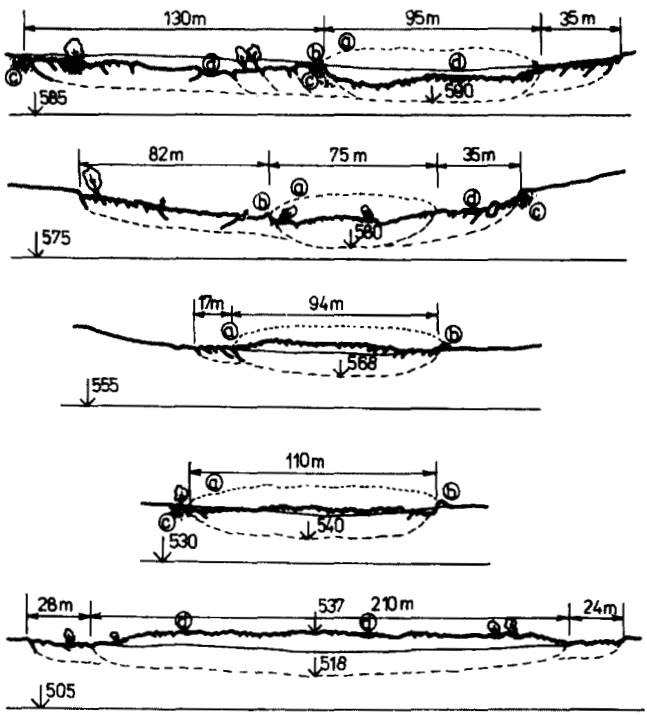


Fig.207 Cross-sections through the landslide near Handlová (profiles denoted as in Fig.208) (after ZÁRUBA and MENCL, 1969).

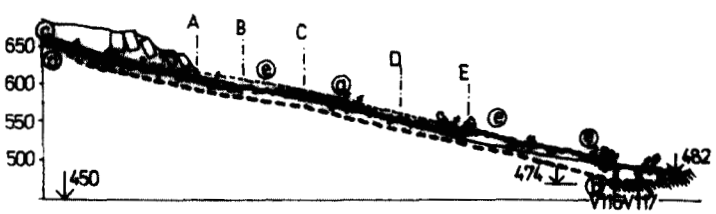


Fig.208 Longitudinal profile through the landslide near Handlová: a – the highest altitude point of the landslide, b – Paleocene substratum, c – colluvial deposits, d – substratum of clays and tuffites, e – lakes on landslide surface (after ZÁRUBA and MENCL, 1969).

The described range of causes of landslides gives an idea of the variety of the effects and allows one to use different landslide classifications (Tabs. 77, 78), ranging from the oldest classification system proposed by HEIM (1882) through the classical classification by Terzaghi (1925) up to the current theories which also take into account the geological and orographic conditions.

Table 77

Classification diagram of slope movements (slipping, flowing) (after NEMČOK et al., 1974)

Basic groups of slope movements	Basic types of slope movements	Examples of the most widespread types and their characteristics	Names of the resultant slope failures
Sliding	Slipping along rotary sliding surface		Rotary landslips
	Slipping along planar sliding surface	Sliding of earth along planar sliding surface	Planar landslips Landslips along planar sliding surface Landslips along predestined sliding surface
	Sliding along compound sliding surface	Sliding along compound, curved and planar sliding surface	Rotary-planar landslides along a compound sliding surface
		Sliding along horizontal or slightly inclined sliding surface	Lateral shifts
Flowing	Flowing of slope deposits	Flowing of clayey and loamy-sand earths	Earth flows Mud flows Earth flows in sensitive clays Solifluction flows
		Flowing of loamy and rubble earths with flood waters	Stone-flood flows Stone-and-mud flood flows Loam-and-mud flood flows
		Flowing of the surface parts of the cover formations during snowmelt periods or as a result of excess precipitation	(flowage)

8.1.1 Movement of surface deposits down the slope

This movement involves the surface strata of the ground in the affected territory. It includes the so-called creeping movement, which occurs in different seasons of the year as a result of typical effects. As the surface layers freeze in winter, the earth mass enlarges its volume and earth particles heave. However, when the earth thaws, these particles do not return to their original place and move farther down on the slope owing to the action of gravity forces. This movement represents only several millimeters annually, but within the horizons of geological eras its importance is great. The creeping movement also influences the underlying rock: friction on the layers' contact surfaces abrades the subsurface layers and bends them in the direction of the slope of the terrain. This action can be traced over long geological periods.

Table 78

Classification of slope movements (collapses) (NEMČOK et al., 1974)

Basic groups of slope movement	Basic types of slope movement	Examples of the most common types and their characteristics	Resultant slope disturbances
collapsing	shedding	displacement of small fragments of semimineral rocks and earths by rolling down the slope	crumbling shedding
	fragment shedding	abrupt displacement of fragments of rocks by free fall and shifting down the slope	shed fragment cones talus cones dumps taluses stone fields stone falling
	bank collapsing	abrupt displacement of rock walls, mostly by free fall	rock collapse bank collapse banks
	planar collapsing	abrupt displacement of rock walls – combination of sliding movement along a predestined path with free fall	sliding planar rock collapsing rock collapsing combined with shifting faults

8.1.2 Stratified landslides

Stratified landslides are characterised by the movement of thin slope deposits (loam, weathered rock) on the underlying rock. The thickness of these layers is between 2 and 3 m. The level of the movement may vary with the location on the slope. The movement proceeds progressively up the hill, starting from the lowest location (chain reaction). The toe of a steep slope, its front, is only stable when the weather is dry. The movement usually resumes in spring.

8.1.3 Liquefied earths

Liquefied earths occur under similar conditions to the landslides described above. Their movement is triggered by intense rains in the headwater parts of large watersheds where weathered materials lie on sloping bedrock. These deposits become saturated with water and flow over the bedrock down to low-gradient locations or to places where the material drains and stabilises.

8.1.4 Liquid deposits

Liquid deposits develop if water infiltrates rapidly through the layers of the earth. The deposits that develop in channels create conditions for the rise of "mures" (mud flows), i.e., for the outwash of weathered rocks together with vegetation after flood rains or after abrupt warming. Liquid deposits occur as a rule above the level of the upper boundary of forest in fissures filled with stone chips and with deposits of fine sands. Slippage of such material involves whole areas and causes extensive, even catastrophic, losses affecting railways, highways and farmland. Dejection cones are saturated with water during flood rains and the deposit loses its state of equilibrium. The summer flood rain recorded in July 1933 in the Tatra Mountains, Slovakia, can serve as an example: the rainfall measured during two hours was as high as 45.8 mm (ZÁRUBA and MENCL, 1969).

8.1.5 Landslides caused by sand washing

This phenomenon is associated with the so-called quicksand. As water flows through the sand, the pore pressure exceeds the gravity of the particles which thus start moving. Conditions are critical where the slope gradient is greater than 1. The critical gradient most frequently occurs with a disturbance of sand strata with a strained groundwater table where a water-bearing layer of sand on an impermeable bedrock is covered by less permeable materials (Figs. 209, 210 and 211).

Fig.209 Effects of water pressure in the soil conducive to the formation of quicksand.

Fig.210 After removal of the material when a cut was made in the lower part of a slope, groundwater formed a spring and as water escaped from the subsurface strata an extensive landslide developed: 1 — impervious bedrock, 2 — sandy earth, 3 — surface loams.

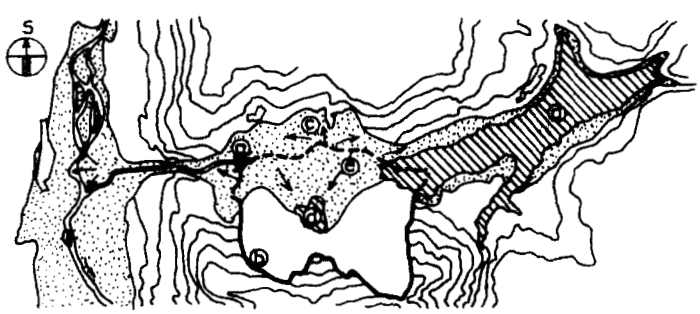


Fig.211 Landslide near the Vaiont Dam, a site plan: a — the dam, b — the detachment line, c — terrain devastated by water and the air-pressure wave, d — lake, e — landslide boundaries (after SELLI and TREVISAN, 1964).

8.2 Methods of control and management of areas exposed to the danger of landslide

If areas endangered by land slippage are to be reclaimed, effective preventive measures need to be taken to stabilise the landslides and a detailed project of interventions and construction work has to be prepared. This will require, first of all, a study of the geological substrate, the properties of the rocks, the hydrological regime of the territory and the effects caused by human activity. It is necessary to gather all pertinent information on the factors and effects that can trigger landslides if a territory is to be reliably protected against landslides at any time in the future.

8.2.1 Field investigations

Field investigations are based on field visual inspection during which the changes observed are recorded in contoured maps to a scale of 1 : 5000 (at the maximum). Ground and aerial photogrammetric methods can be used with advantage for these purposes; common geodetic methods (tachymetry and levelling) will be useful in other cases.

During the investigation it is necessary to carefully record any changes in the terrain (any break or crack, any new springs of water, etc.) and to monitor the extent of the movement in time (Fig. 212).

A cross-section through the landslide's profile is drawn on the basis of geological survey. The marking indicated in Fig. 206 is normally used to characterise the slipping territories.

Geological mapping also has to be performed in areas where extensive engineering efforts are planned. Extensive interventions in the landscape may cause considerable changes in the territory's equilibrium, which has been developing over long periods. Even during an ordinary inspection on foot, if the terrain is closely watched, is it possible to identify landslides that happened in the past: the shape of the trunks of trees, irregularities in otherwise regular tree lines, irregularities in topography and other signs can serve for landslide identification (Fig. 213).

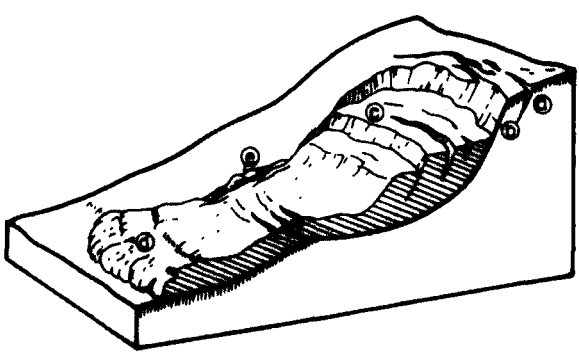


Fig.212 The main parts of the landslide and the form of characteristic cracks: a – top cracks, b – detachment head, c – transverse cracks, d – lengthwise cracks, e – side walls (after VARNES, 1950).

Hydrological survey is among the most important measures of the complex of efforts to stabilise landsliding territories. The project of any stabilisation measures requires perfect knowledge of the territory's hydrology. The hydrological parameters that have to be investigated include the elevation and fluctuation of the groundwater table, direction of subsurface water currents, occurrence of springs, infiltrations, waterlogged soils, etc. Importance is also attached to the location of depressions without water outlets, to tense groundwater tables, permeability of different layers, etc.

The moving layers of rock and earth change the conditions for subsurface water currents, displace the water springs and change water concentrations. The sliding surfaces are usually impervious and the surface and subsurface water that infiltrates through the profile concentrates on them. This gives impetus to the landslide movement and to the rise of new springs.

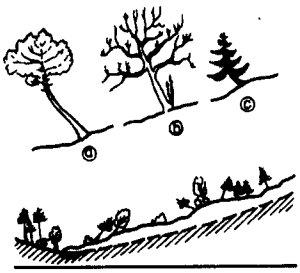
Fluctuation of the groundwater table can be monitored in existing wells or in special probes drilled in the whole area of the landslip territory. Extraordinary attention must be paid to artesian waters that can penetrate to the landslip territory. Piezometers and tensiometers are used for the measurement of water content in the soil profile.

Considering the thickness of the landslip layer, landslips up to 1.5 m thick are considered to be surface landslips. Shallow landslips do not reach deeper than 5 m. Deep landslips may be up to 20 m thick and landslips that are deeper than 20 m are classified as very deep. Probes or trenches have to be dug in the landslip earth to see of what layers it consists. Commonly available equipment and machines are used to speed up the earthwork. Open probes enable good visual inspection of the landslip profile and allow of taking intact earth samples needed for the laboratory analysis of the properties of the material. The probes are, as a rule, installed at stabilised sites (showing no movement). Precautions have to be taken to prevent the machines from triggering further movement of the slip.

The digging of the probe trenches usually proceeds up the slope across the contour lines: as such, they can also be used to convey subsurface water away. As the landslip resumes its movement, the open ditches usually close again. Tunnels and galleries are built for deep drainage. It is recommended to use drilling rigs in field inspections and in installing drains: the drilling equipment will also enable an accurate determination of the depth of the sliding layer under the surface of the terrain.

The slip track and slip velocity are monitored using checkpoints distributed over the landslip territory. The network of the checkpoints is combined with a system of fixed points, located in the stable part of the territory. The points have a regular spacing and their relative movement is recorded in the layout plan. In addition to the surface points, other control points are also located at different depths within the soil profile, using elements installed in the drill holes (e.g., drain tiles): the extent of the movement is then determined after digging an open probe along a continuous cable put into the tiles and having a sufficient portion above the ground (Fig. 214).

The measurement of tension inside the soil profile is an important factor in assessing the danger of landslide. Vertical probes serve this purpose: the deformation of the original circular profile by the stress in the soil is measured inside the



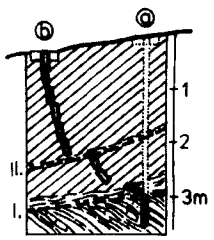


Fig.213 Vegetation cover damaged by previous landslides in the area (after ZÁRUBA and MENCL, 1969).

Fig.214 Method of vertical monitoring of the movement of the separate layers: a — borehole lined with drain tiles on a steel rope, b — distribution of the tiles as a result of the movement of the layers (after opening the probe along the rope) (after ZÁRUBA and MENCL, 1969).

probes. Sensors which enable one to translate the length changes into electric characteristics are used (tensometers). Another method is to use the photoelectric properties of some materials. A hollow cylinder of such material, whose size corresponds to the diameter of the drilled probe, is inserted into the probe to a specified depth and the changes in tension are then visually inspected on a screen (the image is transmitted above the ground). These methods are applicable up to a depth of 10 to 20 m. Holes drilled from underground tunnels and galleries are used for treatment to greater depths.

The recorded data have to be interpreted, preferably by statistical methods. The input data usually vary greatly, so it is often necessary to specify the range of applicability of the characteristics for the probability of occurrence required.

It is also important to determine the properties of rocks using samples taken by means of hollow drills. Other important data to be determined include those on the characteristics of noncohesive soils and deposits and on their texture which has a great influence on their internal friction angle and permeability for water. In cohesive earths it is necessary to determine the pore pressure of water. The methods of measuring and calculating these basic characteristics are treated in literature sources dealing with the mechanics of earths.

8.2.2 Interpreting the results of investigations

After the field investigation is finished it is usually necessary to decide within a short period of time how the endangered area is to be stabilised. ZÁRUBA and MENCL (196) specified two categories of issues to be solved within the shortest possible time, including:

1. project of the cuts and their fillings and the areas of possible slippage that are stable at the moment,
2. stabilisation of the existing lanslides.

The choice of the possible methods for providing scope for conveying water out of the soil is important in the economic evaluation of the result to be obtained. It is recommended to use suitable machines for digging the cuts: much earthwork is usually involved. The construction of shafts, tunnels and galleries is usually very

costly, so it is advantageous, everywhere possible, to replace them by horizontally drilled holes.

The engineer has not only to pick out the cheapest option but must also take into account the local conditions, disturbing climatic effects, etc.

It is important to assess the form of the sliding surface. The following situations are usually encountered:

1. the origin of the sliding surface relates to the geological conditions of the territory – flat bedrock, original sliding surface of an old landslide.
2. flat sliding surfaces occur under sandy soils and stony deposits – the slippage continues until a state of equilibrium is attained,
3. in the case of cohesive homogeneous soils, the shape of the sliding surface is cylindrical; the cross-section is comparatively close to the form of a logarithmic spiral.

A whole range of methods in the mechanics of earths is used to assess the tension in the strata of the earths (Petterson's method, Bishop's method and others).

8.2.3 Securing the landslide territory

The interventions to be carried out in landslip territories should proceed as follows (listed in their order of importance):

a) intercept and convey away all the surface waters entering the landslip area and the water (if any) springing in the area of the landslip head,

b) pump water from all the wells inside the landslip territory and remove water from the terrain depressions,

c) fill and seal all open cracks which allow water to penetrate from the soil surface to the slipping surface.

When these works are finished, the next step is to install subsurface drainage (separate drains), to build galleries (or drill horizontal holes), and finally adjust the surface of the terrain to the intended future management system.

The time scale of the works must be prepared with due respect to the climatic conditions of the area. It is usually impossible to work in winter and spring because of the problems of diversion of surface water. When planning the work, it usually has to be borne in mind that the work cannot be just started and then interrupted: if necessary, it is better to postpone the work to a later period.

Different measures can be taken to stabilise slopes. For example, stability can be improved by alleviating the load in the upper part of the slope of by adding weight to the slope's toe. ZÁRUBA and MENCL believe that moving 4% of the potential slipping mass from the upper part of the slope to the toe part will increase stability by 10%. If it is dangerous to take material from the top of the slope to dump it at the slope's toe, it is possible to take it from a suitable material pit. Such an intervention cannot be used at sites with poorly drained bedrock where the additional weight may impair, and not improve, stability. Slope toes cannot be protected by additional weight along roads or railways cut in the slope or running along a canal. In such cases, the only solution is to remove part of the weight (material) from the top.

Drainage of failed slopes is the key stabilising measure. The territory affected by a landslip is usually wavy, waterlogged and divided by numerous transverse cracks.

Water concentrates in the depressions, penetrates into the earth and saturates it. Surface drainage of the landslide area is therefore the first thing to do. Though surface drainage can hardly provide sufficient stability of a territory in motion, it can significantly contribute to stopping the motion.

One of the first interventions is to intercept the temporary small springs and streams that enter the area affected. The intercepted water must be safely conveyed outside the endangered area. The same applies to the springs inside the endangered area: they have to be opened and intercepted as high on the slope as possible. Any available portable pipe systems can be used for this purpose. Portable irrigation pipelines with quick couplers, air conditioning pipelines, nonperforated flexible drain pipelines (diameter 100 mm – can be ordered from the manufacturer), fire-control hoses and other such equipment can be used. It is an advantage of temporarily installed pipelines that they can be quickly repaired if the movement of the terrain damages them. Of course, problems arise if surface pipelines are to be used in freezing weather.

Partial stabilisation is followed by digging open ditches, having a suitable gradient and flow profile, to convey precipitation water. The surface of the terrain is then adjusted: barriers and depressions are levelled and cracks are filled to allow water to run freely to the interception ditches. The surface levelling work should be performed carefully so as not to cause too much damage to the soddy surface of the terrain because a denuded soil surface is less resistant to erosion and allows water to penetrate to the impermeable bedrock. The slopes and the bottom of the ditches have to be lined to resist the destructive effects of flowing water and to prevent water from penetrating underneath. Any common lining method can be used: in sandy soils both the bottom and the walls of the ditch can be protected using bitumen or asphalt, possibly also oil spraying. Prefabricated plastic lining can also be used. In heavy-textured and medium heavy soils, the lining should be laid onto a sand bedding: water seeps from the soil to the bedding which conveys it in the direction of the ditch. The bottom of the ditch has small "drops" into which the water seeping through the bedding is directed using drain pipes. If shaped channel-lining blocks are available, they can be used with advantage: they should be laid in a stepwise manner, starting from the lower end of the ditch and proceeding upstream, the end of each prefab part being laid onto the preceding one. Water from under the channel can penetrate freely into the ditch and the channel lining is able to adapt to the possible partial movements of the terrain. Interception ditches also have to be built outside the affected area to prevent water from external sources from penetrating into it. The ditches must be lined and watertight, but should allow water to penetrate from the sand bedding to the channel: they must have the same gradient along their whole length to prevent the transported fine particles from depositing in places with a reduced tangential stress. The channels are usually shallow and if they become clogged the water could spill over the surface of the endangered terrain. Under normal conditions the width of the channel bottom is about 0.5 m and depth ranges from 0.3 to 0.6 m. To convey the concentrated runoff in the end part of the drainage systems, chutes are usually built, often very steep, which should have a well-dimensioned stilling basin at the end.

Groundwater is the main cause of the nonstability of sloping terrains. Drained slopes, though comparatively steep, have a higher stability.

Systematic drainage can only be built after thorough geological and hydrological investigation. The first intervention aimed at reducing the water table in the terrain is to pump water from drill holes. The hole should be wide enough to accommodate a casing with a filtration filling around it, and the diameter of the casing, in turn, should be large enough to allow a submersible pump to be put inside. If old wells are located in the area, water has to be pumped from them. During the early stage of the work, portable fire pumps or irrigation sets can be used to empty the wells.

Deep drill holes will also allow of seeing which geological strata the slope consists of. Having analysed the geological composition of the bedrock, the definitive drainage interventions can be designed. Systematic drainage is built on the surface and horizontal bores and galleries will provide drainage in the deeper strata.

Galleries are designed to intercept the water that infiltrates through the rock and to solve the associated hydrological problems. Galleries are, in principle, tunnels cut at a gradient increasing in the direction in which the draining proceeds. Additional drain bores in the gallery's floor, sides and roof will provide additional draining potential.

If water penetrates into the gallery during the work, a ditch has to be dug in the gallery floor to convey the water away. The gallery may be built under the slipping surface; in such cases, vertical holes have to be made in the gallery roof to allow water to get from the slipping surface into the gallery. The cross profile of the gallery should be sufficient for its draining function to be maintained though the walls may be partially damaged.

Galleries are effective and offer a number of advantages. However, on the other hand, they have some drawbacks, including high investment costs, labour intensity of construction and little scope for the use of machines. There is a great danger of sinking during the construction. The discharge from the gallery is often only several litres and the profile of the tunnel is several orders larger than needed. The gallery should not remain empty and must be filled with permeable material (stone, gravel). It is currently possible to make galleries even longer than 200 m.

Horizontal boreholes are much less expensive: compared with the galleries, horizontal holes require five times less investment and the time needed to make such a hole is much shorter. On the other hand, hitting the impermeable bedrock which intercepts the water and is responsible for the nonstability of the slope can hardly be guaranteed. The boreholes cannot be longer than 200 m. On slopes of a low gradient, short drain drill holes are installed by driving perforated pipes into the slope. This is a good solution for locations where the water-bearing bed of gravel or sand is comparatively low.

Augers are most frequently used to drill long drain holes. Perforated casings (pipes) are inserted into the drill holes. This method is used with advantage in rocks of poor cohesion. Drilling bits of different shapes are used to make bore holes in harder rocks. As the hole is drilled, perforated pipe (acting as casing) is pushed into the hole. The drilling bit will remain in the finished hole. The casing should be at least 6 mm thick and the inner diameter should be more than 84 mm, otherwise they may be deformed during the course of the drilling. It is advantageous to start the drilling with a small upward incline in the drilling direction: this will prevent cold air from getting into the drill hole in winter and freezing the water at the mouth or even inside the hole itself. It should also be borne in mind

that though the hole may be inclined upwards at the beginning, the weight of the auger and inserted piping will bend it down. The end of a hole 60 m long may be 2−3 m below the level it would have, were the original direction maintained.

Horizontal drain bore holes at the toe of the slope or at the lower end of the slipping surface do not guarantee the stability of the terrain. If there are several water-bearing layers on the slope, horizontal bore holes should also be made in the upper part of the slope.

Drain ditches and trenches are among the oldest draining interventions in landslide territories. If they are to have a sufficient draining effect, they should be cut to sufficient depths, depending on the geological conditions of each particular site: if they do not reach down to the impermeable bedrock, their effect will be poor and the slippage may continue. Ditches and trenches are filled with different materials such as brushwood, other organic materials, gravel, rock, etc. The digging is usually costly because casing is needed. If the soil is cohesionless, the walls of the trench or ditch often threaten to sink. They take a long time to build, so it often happens that they cave before all work is finished. The digging should proceed against the direction of the slope to allow the penetrating water to flow away freely. As the flowing water carries soil particles during the digging work, it is not possible to fill the trench continuously as it is built: the material suspended in the water would settle in the ditch and reduce its draining effectiveness. Hence, the whole length of the ditch or trench must be finished before the filling with permeable material can start. If the impermeable layer is deep under the surface of the ground, it is better to build a gallery or make a horizontal borehole. Deep trenches and ditches will be useful in locations where the impermeable layer does not lie so deep underground. The trench should run across the gradient of the slope and should be cut into the impermeable layer. If dug in this way, it is able to intercept the water and to concentrate its runoff from the potential slipping surface.

If this intervention is to be effective, the trench should reach below the freezing depths of the soil. The trench's filling is separated from the soil surface by a humus layer laid onto an impervious or filtrating layer (PE film, reversed gravel filters, layers of loppings, straw, etc.) which prevents the filling from clogging. In stratified sandy earths between which thin layers of impermeable material are sandwiched, the bottom of the trenches has to be protected to prevent infiltration of water to the bedrock layers. If water penetrates into the low layers it might trigger slippage of the upper layers on the impervious gliding surface of the bedrock stratum, thus causing even greater losses. For an appropriate design of the construction of trenches and ditches, it is necessary to know in detail the geological structure of the earth's surface in the landslip area.

Large amounts of earthwork are involved in such efforts, therefore powerful equipment is now commonly used. Of course, the size of the trenches built with the use of machines is larger and much more filling material is needed; however, on the other hand, the effectiveness of such interventions is great.

Landslides usually destroy much of the vegetation cover in the area affected. Once the terrain is stabilised by the technical interventions, the vegetation should be regenerated. The direct stabilising effect of the vegetation is low because the roots of plants and trees do not reach very deep. Greater importance is attached to vegetation's indirect effect − reduction of the amount of water infiltrating into

the deeper strata of the landslip. On the other hand, forest stands that grow fast and produce a considerable load on the landslip surface bear many disadvantages. Sýkora (1961) recommends first to develop a compact vegetation cover and then, once the plant cover is well established, to plant the trees.

8.2.4 Special structural approaches to landslide stabilisation

Pipelines, cables and especially roads and railways that have to cross areas endangered by landslides need special attention. Some need excavations, for others embankments have to be built. In both cases the original stability of the earth is disturbed. Retaining walls have to be built to protect the structures and the endangered area itself should be protected with reinforcing ribs (Fig. 215). Measures should also be taken to protect the construction against the effect of frost and infiltrating water.

If there is sufficient space, it is recommended to build the retaining wall at some distance from the toe of the slope (from the lower end of the slipping area) and to fill the space between the wall and the slope with earth to improve the slope's stability.

It is often necessary to remove much earth and rock when building a road or railway. Weathering of the denuded rocks may then loosen individual rocks or lead to the danger of failure of entire rock layers. In such cases, the loosened layers have to be tied or metal nets have to be spread over the weathered slopes. Anchors may be used to tie the rocks and to fix the nets. Steel bars or steel-wire cables make good anchors: on the side located in the rock, they are fitted with an anchoring element and on the other end they have a steel rod with a thread. The anchor is fixed in the rock and embedded in cement mortar; then a plate is fitted on the rod and the anchoring system is then prestressed, using a nut. The anchors have to be painted or coated with plastic film to control corrosion.

Piles can be useful in stabilising shallow layers in landslide terrains. The piles increase friction, thus improving the stability of the slope. Piles are ineffective in plastic soils because if such soils slip down they just pass by the piles. If the slipping layers are thick, the effect of the piles is problematic. They may be useful for temporarily containing the pressure of the earth while a road or railway is being built, the spacing of the piles being 1 to 1.5 m. A borehole is usually prepared for each pile to be driven in.

When stabilising landslide areas, it may sometimes be necessary to use special methods to consolidate the foundations of structures.

Electroosmosis is among the key methods. Perforated steel tubes (diameter 200 mm) are, as a rule, used as electrodes. The tube also serves as casing in which

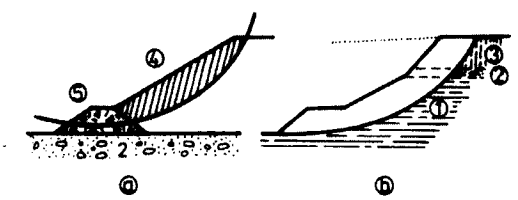


Fig.215 Landslide form (b) and the method of its control (a, b): 1 — Neogene clay, 2 — gravel sand, 3 — loam, 4 — made-up ground, 5 — buttress at slope toe.

a submersible pump is installed. The method is described by ZÁRUBA and MENCL (1969): a system of tube electrodes was built, the electrodes being located at the apexes of quadrangles (sides 10 m long) in two rows. The electrodes (casings) were alternately connected as anodes and cathodes. When they were not connected to electric current, the amount of water removed by pumping from the casings (cathodes) was 50 litres per hour but when they were connected (40 V), the pumping performance increased to 479 litres per hour. The consumption of electric energy was 0.4 kWh daily per 1 m^3 of excavated space (the area of the foundation pit was 230 × 160 m).

Another method is thermic stabilisation developed by LITVINOV (1955). The method is based, in essence, on burning a vertical cylindrical jacket around a drilled hole (diameter 0.1 − 0.2 m). This is effected by blowing hot gas (about 1000°C) into the hole. The gases are produced by a burner; air is blown at a pressure of about 0.015 to 0.05 MPa into the flame. The burning effect covers a range of 1.0 to 1.5 m round the hole. In clay soils where the pores have a very small diameter, two holes have to be bored: one to blow the hot gases in and the other to serve as an exhaust pipe for the cooled gases to escape. In such a case, the gas can escape under its natural pressure along the other tube (acting as a chimney) and no compressor is needed.

The most advantageous method of the reinforcing of the earth is grouting with cement slurry: a system of grouting tubes (diameter 50 mm) is installed in the earth or rock and the slurry is forced into the tubes. A solution of clay can be used instead of the cement slurry. Grouting can only be practiced in soils permeable to the grouting material, i.e., those whose pores are about 0.4 mm in diameter. The grouting may proceed by steps: after drilling the bore to the required depth, the slurry is forced into the bottom part of the hole, then the treated level is sealed and the next (upper) layer is treated − and so on up to the surface. The reverse procedure is also possible: the upper layer is treated first and when it hardens the hole is deepened to treat the layer underneath. This method is advantageous in less cohesive soils because it eliminates the need to use casings. Grouting apparatus is used to force the slurry into the hole.

The same technology is used for the application of chemical solutions. Such methods use the action of silicic acid gel which is produced when silicic acid comes into contact with silicate solutions. The gel develops into solid and impermeable material. Sands and loess are reinforced (and their permeability is reduced) in this way. It is also possible to use a solution of water glass or calcium chloride. Grouting apparatuses consisting of seamless pipes (diameter 19 − 38 mm, wall thickness 5 mm) are used to apply the solutions. Injection holes 1 to 2 mm wide are located on the pipe behind the tip of the needle. The needles may be driven to a depth of 15 m.

ZÁRUBA and MENCL also describe a method of using explosives to check the slipping movement of the earth. The purpose is to break and roughen the sliding surface, loosening the explosion zone. This improves the conditions for conveying water to the lower horizons or for entrapping it in locations where it cannot contribute to the slipping effect. The method is ineffective in fine-grained earths in which it has no influence on the stability of the terrain. The favourable effect in the stabilisation of landslides usually does not last long (Fig. 216).

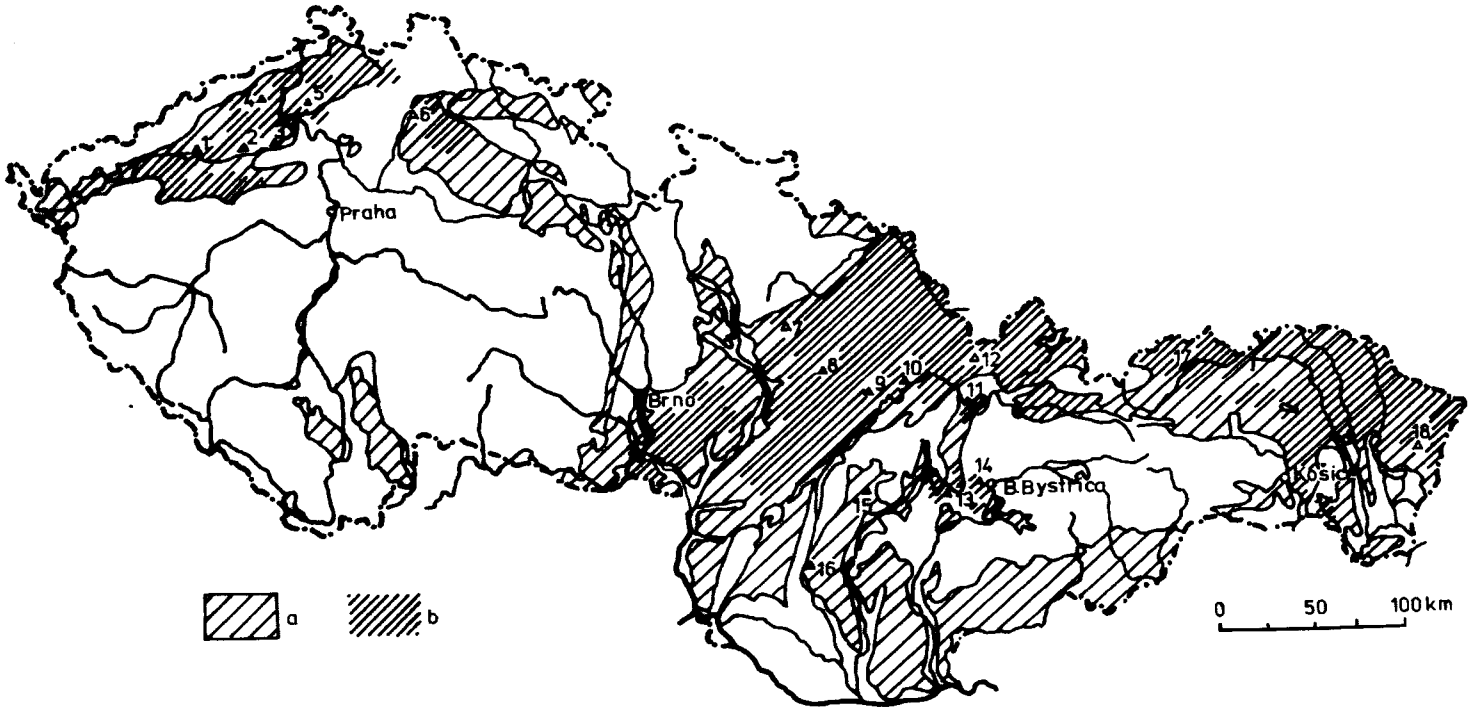


Fig.216 Modified map of landslide occurrence rates (after MATULA et al., 1963).

CHAPTER 9

Vegetation in the Landscape

9.1 Forests in the landscape

Forests are among the greatest riches of the majority of countries: they constitute a substantial environmental feature and a renewable source of timber for the economy. Forests influence and improve the climate and the hydrological and soil conditions, provide a natural habitat for many species of animals and plants and their associations, conserve the natural beauties and are a source of health and recreation for the people. For all these and many other reasons, people have to protect the forest fund, including the forest land and the woody species growing on that land. Planned and systematic efforts have to be made to improve the forest stands and to manage them by carefully applying the best of biological, technological and economic knowledge.

From the viewpoint of their prevailing functions, forests are divided into three categories, including commercial forests, protection forests and special-purpose forests.

Inclusion in any of these categories indicates what general purpose society gives to each particular forest and shows, at the same time, in what ways the forest is to be managed. The purpose of the forests in the protection category is clearly defined by natural conditions. Those in the special purpose category have specific missions.

Commercial forests. Their main purpose is to produce raw timber and exploit minor forest produce, and also to practice game management. The "minor forest produce" includes all other commercial activities than the production of timber: exploitation of bark, collection of drug plants, picking of mushrooms, forest fruits and forest seeds, forest grazing, exploitation of forest litter, bitumen collection, quarrying and exploitation of sand. Game management in commercial forests and fishing in forest waters also belong in this category – both these activities are of great importance for recreation and wildlife protection.

Protection forests. Their purpose is associated with the natural conditions. Their management is focused on their protective mission. The following forest types are included in the category:

a) forests in very unfavourable locations such as scree sites, gullies, ridges and hillsides with compact outcrops of parent rock, nonstabilised gravel deposits and deep peat bogs,

b) high mountain forests below the boundary of tree vegetation protecting the forests at lower altitudes, forests on mountain ridges exposed to severe climatic effects and forests controlling the danger of avalanches,

c) forests in the mountain pine zone,

d) forests needed for soil conservation.

Special-purpose forests. Their mission ensues from the specific requirements of over-all societal importance which also determine the management practices in such forests. This category includes:

a) forests in the protection zones of water sources,

b) forests in the protection zones of natural medicinal resources and natural mineral-water sources, spa forests and forests around preventative medical-care facilities,

c) forests in registered preserves and pheasantries,

d) forests in National Parks and Protected Landscape Regions,

e) forests heavily affected by industrial exhalations and requiring special management approaches,

f) forests intended for forest research,

g) forests in which important requirements of the society imply the need to use special methods of management (forests in the vicinity of cities, forests built-up with weekend chalets, military forests, school forests and others).

According to their formation, tending and regeneration, and also according to the production cycle, the following forest stand types are recognised: high forest, coppice forest and composite forest (coppice with standards).

High forests either have grown from seeds (either sown by man or self-sown) or were planted. Their production cycle is long (100 to 140 years). Trees in such forests grow to a large size and produce tall trunks of a large diameter. These are the best-quality forests of the greatest economic value and efforts are made to increase the proportion of this forest stand type. Of course, not all forests can be transferred to the high forest category because some sites require the other types.

The coppice forests developed by sprouting from stumps or roots. Such trees grow quickly up to a certain age because they are supported by a large root system and have a rich source of nutrients in the stump. However, at an age of 30–60 years their increment declines, so these forests do not reach the height of those in the high forest category.

The decline in increments – and also in sprouting capacity – marks the beginning of the time for felling. The rotation periods of oak coppice forests are 40–60 years, alders 30–60 years, locust 10–30 years, willows 8 years, etc.

Coppice forests produce wood of poorer quality (usually fuelwood) within a comparatively short period. This ancient stand type requires simple management practices at minimum costs. At present, coppice forests are preferred only at sites where their performance is better, compared with high forests. This primarily applies to special-purpose forests and mainly protection forests, riverine forests, willow growths, etc.

Composite forest with standards is a combination of high forest and coppice forest at the same site. The middle layer of the forest produces fuelwood and the upper layer (standards) yield commercial timber. It is a characteristic feature of this forest stand type that strong trees of seed origin, called reserve standards, develop in such a forest within two or three or more production cycles of the coppice. The coppice usually consists of shade-loving deciduous species (maple, hornbeam, linden) and the reserve standards include sun-loving deciduous and coniferous species (ash, beech, oak, larch, pine and others).

9.1.1 Functions of forests

Forests have the following functions in the landscape: timber and wood production, water conservation, maintaining microclimatic and hygienic conditions, soil conservation and improvement, providing conditions for recreation, and aesthetic, cultural and other functions.

9.1.1.1 Water management function

Vegetation has a favourable influence on the water regime and water balance. The aerial parts of plants intercept precipitation and slow down its incidence onto the soil surface, thus preventing it from breaking up the soil particles. The root systems of the plants interact with the humus layer in influencing the infiltration of water into the soil, thus directly acting on the division of total runoff into underground and surface runoff. Of course, different plant associations have a different performance in this activity (Tab. 79 to 81).

The water-management effect of forest can be divided into two major parts:

a) qualitative: in a broader sense, this covers the influence on the course of the runoff of precipitation water, i.e., reduction of the fluctuation in flow rates, reduction of maximum flows, increase in minimum flows, prevention or reduction of erosion, etc.,

Table 79

Evapotranspiration of forest stands in mm of precipitation (after MOLCHANOVA, in RIEDL et al., 1973)

Species	Annual precipitation	Category of water demand	Age of the trees							
			20	40	60	80	100	120	140	160
Pine (*Pinus* L.)	550	transpiration	236	250	200	186	170	158	146	125
		evaporation from soil and soil cover	48	67	87	100	100	100	103	105
		interception	127	150	140	135	120	105	100	97
		total	411	467	427	420	390	333	349	327
Oak (*Quercus* L.)	523	transpiration	310	342	352	323	289	263	255	252
		evaporation from soil and soil cover	65	78	84	90	94	98	104	104
		interception	49	63	64	60	58	58	60	60
		total	424	483	500	473	441	419	419	416
Spruce (*Picea A.* DIETR.)	480	transpiration	203	291	300	278	219	193	188	188
		evaporation from soil and soil cover	60	55	55	60	58	75	80	83
		interception	158	175	185	180	170	160	144	128
		total	421	521	540	518	447	428	412	399

Table 80

Interception, water flowing down along the trunks and precipitation water reaching the soil in a spruce and beech forest (after EIDMAN, 1959, in ZACHAR et al., 1984)

Species	Season	Precipitation		Interception in crowns		Water running down the trunk		Water reaching the soil	
		mm	%	mm	%	mm	%	mm	%
Spruce (*Picea A.* DIETER)	summer	629	100	196	31	5	1	433	69
	winter	587	100	118	20	4	1	469	80
	year	1216	100	314	26	9	1	902	74
Beech (*Fagus* L.)	summer	629	100	68	11	104	16	561	89
	winter	587	100	25	4	97	17	562	96
	year	1216	100	93	8	201	16	1123	92

Table 81

Surface runoff in afforested and open watersheds (after LHOTA and MAŘAN, 1947, in ZACHAR et al., 1984)

Spruce stand			Pasture		
Precipitation	Surface runoff		Precipitation	Surface runoff	
$mm \cdot h^{-1}$	$m^3 \cdot s^{-1} \cdot km^{-2}$	% of precipitation	$mm \cdot h^{-1}$	$m^3 \cdot s^{-1} \cdot km^{-2}$	% of precipitation
15.5	0.092	2.13	15.5	0.882	20.25
29.5	0.297	3.69	31.0	2.672	30.68
44.5	0.535	4.30	40.4	6.056	55.76
55.5	0.618	4.08	50.8	8.172	57.91
79.5	0.900	4.06	65.6	10.500	57.72
88.0	1.330	5.44	94.8	17.445	66.27

b) quantitative: this effect generally covers the influence of vegetation on the amounts of water available for various uses.

The qualitative effects of forest include:

– hydric effects: buffering the flood waves from snowmelt and flood rains (distribution of the runoff over a longer period), reducing the fluctuations in water levels in streams, etc.,

– hygienic effects: the quality of the runoff water depends, besides the physical and chemical composition of the soil strata through which it passes, also on the occurrence of chemicals that penetrate into the soil (mainly agricultural chemicals) and on the quality of atmospheric precipitation. Forestry uses much smaller amounts of chemicals and much less waste usually pollutes forest land, compared

with agriculture, so the quality of surface water, and especially underground water, is much better in forests than in the forest-free areas.

– soil protection effects: Afforested areas, compared to other types of landscape, have a better potential to resist erosion. This is also associated with the better quality of water (particularly surface water).

The quantitative effects of forests on water management include:

– the effect of interception: the amount of total annual precipitation in different forest stands is 5 to 52% more, compared with bare areas. The higher the altitude above sea level, the lower the effects of interception (at the highest altitude the interception level may even be negative),

– effect of transpiration: it is generally believed that losses through transpiration are the same as those from interception,

– effect on evaporation from the soil: physical evaporation from the soil is not high – about 6–9% of precipitation in the open areas. However, in densely afforested regions the total precipitation is 6–15% more than in sparsely afforested watersheds, which is due to increased interception and transpiration of forest stands.

Compared with forest-free areas, a forest generally influences water circulation as follows:

– it adds horizontal precipitation to the absolute precipitation figure,

– it increases the total value of evaporation by interception and transpiration, though it reduces evaporation from the soil,

– it transfers a large amount of precipitation water into the soil through increased infiltration and increases the accumulation of water in the soil,

– it reduces the level of surface runoff, thus alleviating the potential effects of water erosion,

– it slows down the runoff of water from the watershed, thus reducing the maximum flows.

Considering the mentioned loss items of the water balance, various forest-management measures have to be taken to maximise the quantitative effects of forests and to reduce the losses. For locations where horizontal precipitation is not involved, ZELENÝ (1969) recommends the following measures to be taken:

a) suitable age structure of the stands: stands up to 20 years of age are the best and most effective,

b) selection of species according to the site: species of low interception and transpiration,

c) tending of young stands: thinning the stands to allow the largest possible amount of precipitation water to get onto the soil surface,

d) forest regeneration, especially planting in areas cleared by cutting.

9.1.1.2 Microclimatic and hygienic functions of forests

Vegetation modifies the effects of a number of meteorological factors, e.g., radiation, light, temperature, humidity, evaporation and others. By changing the influence of these factors, vegetation may influence the water regime of the soil, the circulation of nutrients, the life of microorganisms in the soil and other environmental features. Generally, the microclimatic and hygienic function of forests consists in the following effects:

– reducing the fluctuation in temperature and humidity,
– increasing the relative humidity of the air,
– controlling the direction of the movement of the air,
– reducing the amount of dust in the environment,
– reducing the content of carbon dioxide and other pollutants in the air,
– absorbing noise,
– producing etheric oils and having (some plants) a repellent action (phytoncides),
– contributing to the ionization of the air,
– (the green colour) providing rest for the eyes and general relief for the nervous system, etc.

9.1.1.3 Soil-improvement function

Vegetation, especially woody forest species, acts favourably on the properties of the soil. These effects may include:

a) drainage (biological drainage)
b) colmation – sluicing,
c) desalinisation (reduction of physical evaporation),
d) fertility improvement – regradation (pioneer species and/or soil improving species).

9.1.1.4 Soil-conservation function

Soil conservation is the dominant environmental function of vegetation. The soil-conserving effects of vegetation include its favourable influence on soil properties and the soil-protection effect of the plant cover. The general soil-conservation function of vegetation can be subdivided into the following effects:

– erosion control,
– deflation control,
– landslide control,
– bank protection,
– avalanche control.

9.1.1.5 Recreational function

The use of forests for recreational purposes is among their social functions. Man uses the favourable effects of the forest environment for refreshing his physical and psychic strength. Man seeks refreshment in greenery (vegetation) instinctively because during the long course of evolution he has perfectly adapted to the green environment. Verdure gives optimism and acts favourably on man's eyesight.

9.2 Fundamentals of the formation of vegetation

9.2.1 Forest nursery practice

The forest nursery is a piece of land serving primarily for the production of woody forest species' seedlings and plants for artificial regeneration, for afforestation of nonforest areas, for land conservation by forestry methods, for planting alleys, etc.

The following types of forest nurseries are currently established:

a) regional nurseries – the basic type of nursery, usually larger than 15 ha,

b) specialised regional nurseries – their area is usually larger than three hectares. They are established in areas where conditions are not favourable for building a regional nursery,

c) central nurseries – designed to produce plants from seedlings supplied from the central or regional nurseries. Their area usually ranges from 10 to 25 ha,

d) combined and other types of nurseries – temporary nurseries serving until the other three types are able to meet all requirements for planting material.

The total area of the nursery includes several compartments: the production area where the planting material is grown – beds for sowing, transplanting and other forest-plant production methods (container plants, plants from cuttings, etc.), the soil-regeneration area (reserve for the four-year soil-improvement cycle) and the service area (network of permanent paths and roads, manipulation area for the machines, houses with facilities for employees, stores, garages, etc.). The size of the soil-regeneration area is equal to 25% of the production area and the same applies to the service area. The nursery must have a fence 1.5 m tall at the minimum and it must penetrate at least 0.25 m under the soil surface, for protection against damage by animals. The nursery must be protected against penetration of surface waters from outside the nursery area.

9.2.1.1 Seedling production

The term "seedling" covers the growth stages of woody forest species from germination to transplanting, undercutting, or transplanting into containers. All the further growth stages are called plants.

Preparing the soil for seeding

A good quality of seedbed preparation, combined with good fertilisation and treatment, is essential for the production of good planting material and also for effective weed, insect and fungal disease control. The actual method of soil preparation depends on the method of production of the planting material; this applies to both the main soil preparing operations and the final stage just before planting.

The key operation is deep tillage in autumn (0.2 – 0.3 m). Spring tillage is not considered favourable because it leads to the drying of the soil. In spring, it is necessary to smooth the surface with a smoothing harrow. Before sowing, the soil (a shallow surface layer) is loosened (e.g., rotavated) and the surface is smoothed. Just before sowing, the surface of the soil must be slightly compacted (e.g., by rolling), especially if the fine seeds are to be sown.

Sowing

Seeds are drilled in rows (rills) 20 mm wide or in strips 50 – 75 mm wide, or are broadcast. The strips are better than rills because in the latter case the seedlings are closely clustered and their roots grow tangled together. Broadcasting is not

advantageous because the random distribution of the seedlings makes it difficult to weed and cultivate the beds.

Row spacing depends on the species and on the planned length of time for which the seedlings are to be grown. With conifers to be transplanted, the row spacing is 0.1 m, in conifers not to be transplanted and in deciduous species to be lifted as one-year old seedlings, the spacing is 0.15 m and in biennial deciduous seedlings 0.2 m.

The seeding rate depends on the species, on the seedlings' growth rate, on seed quality and on further use (transplanting, planting in definitive locations). The seeding rate is given in grams per 1 metre of the row or strip. The rills into which the seeds are drilled are made using different rill boards or row markers and marking rollers. Hand sowing is costly and is only used for pregerminated and small stratified seeds. Various single- or multiple-row machines are increasingly used for this purpose, including different adapters to minitractors. After sowing, the rills are filled with materials which should remain loose, do not hinder the growth of the young plants, are free from properties conducive to drought, have good insulating characteristics, are free from weeds and have the same pH as the earth on the seedbed itself. Such materials include sand, peat, soil from garden beds, sawdust and others.

Seedling production under plastic film covers has become widespread in recent years, mainly for economic reasons and also because of the lack of forest-tree seeds. This method is associated with the production of seedbed substrates enriched with a well balanced nutrient spectrum (peat, bark and others). Hydroponic methods for seedling production have also been developed.

Once in the soil, the seeds require intensive care: in particular, an optimum water and air regime must be maintained in the soil. The following measures have to be taken:

- cover the seedbed, using brushwood, substrate briquettes ("half-balls"), narrow plates, frames fitted with different shading sheets, tilting shades, mats, etc. When the seedlings emerge, raise or remove the covers,
- control drought,
- irrigate when necessary.

The following operations have to be performed after the seedlings emerge above the surface:

a) cover the surface between the rows to prevent drying and to control weeds,
b) shade the emerging seedlings,
c) control weeds,
d) single the seedlings if they are dense,
e) irrigate when necessary,
f) prepare the seedlings (or weak plants that will have to stay on the beds over winter) for the winter season (cover them with peat, leaf mould etc. to protect them against dry frost).

9.2.1.2 Production of plants

Plants with a rich root system, with a sufficiently thick root neck and a branched above-ground part can be produced after transplanting the seedlings or by other methods, e.g., multiple replanting, production of container plants using peat pots, PE bags, or PE disks (Nisul's method). With some species, mainly the deep-rooting ones, the required result can also be obtained by undercutting the roots.

Transplanting

Seedlings have to be transplanted to develop into planting material with an ample root system and adequately developed aerial part.

Depending on the season of the year in which the seedlings are transplanted, we recognise:

– spring transplanting: most common method, the seedlings being transplanted in March to April (at higher altitudes also in May and June). The seedlings have to be pregerminated,

– summer transplanting: an advanced method suitable for some species such as Norway spruce, Douglas fir and others and for seedlings grown under plastic cover. The seedlings are transplanted in late July to late August.

– autumn transplanting: suitable for lowland situations, light-textured soils and some species (European beech, European larch). The transplanting should be completed before mid-November.

The seedlings to be transplanted must be of top quality and of a suitable age and size. Seedlings between three months and three years of age can be transplanted by hand. The best size of seedlings for machine transplanting is 0.1 to 0.15 m for conifers and 0.1 to 0.12 m for deciduous species.

Production of grown-up plants

Half-saplings and saplings are commonly used for the afforestation of heavily weed-infested devastated areas where the trees are exposed to frequent ground frosts and browsing by game animals, and for the afforestation of farmed landscapes (wind breaks, alleys, riparian stands, etc.).

Half-saplings are four- to six-year old plants with aerial parts 0.5 to 1.3 m tall (usually transplanted twice).

Saplings are six years old or older plants with a shaped crown, 1.5 to 2.5 m tall. They must have been transplanted twice to four times, as necessary.

Production of container plants

This is an advanced method of growing forest plants. Seeds or seedlings grown under plastic film or in an open area or plants that developed from self-seeding are planted in a container with substrate. They have to be planted after 1 – 2 years of continuous development. If degradable containers are used they can be left on the plants' rootballs; nondegradable containers have to be removed before planting.

Plants in peat-and-pulp pots. These degradable pots are moulded from peat, pulp and cement. They are made in different sizes and may have the shape of truncated cones or may be angular, and are usually connected into blocks. In the nursery, they are put, with the plants inside, into frames onto a solid plate (so as not to allow the roots to grow into the ground) and are fertilised and irrigated as necessary.

Plants in plastic film bags. PE bags are most frequently used. They are 0.2 m tall and 0.1 m wide and their bottom third is perforated. The perforation makes it possible to regulate temperature and moisture and allows the roots to grow into the surrounding soil after planting.

Many other methods of ball-plant cultivation are used, each at a different level of mechanisation. Plants (seedlings) with rootballs are most frequently used for the afforestation of locations with extreme conditions.

Harvesting the planting material

The main harvesting operations are as follows: lifting, grading and bundling, storing, packing, dispatching and transport.

Planting material production by vegetative propagation

With some woody species it is better to use vegetative propagation: some species may not be readily available; seeds may quickly lose germinative power. The commonest technique of vegetative propagation is the use of cuttings. The major types of cuttings include winter cuttings, summer cuttings and root cuttings.

Winter cuttings are taken in mid-December to mid-February, usually from trees grown for that purpose in rootstock nurseries. The cuttings themselves are cut from longer sprouts in rooms where the temperature is not higher than 12°C. The cuttings should be 0.1 – 0.2 m long and their best diameter is 10 mm. Those to be used for the vegetation lining of water courses should be 0.3 – 0.5 m long and 30 mm in diameter. Each cutting should have 2 – 3 buds and should be cut with a garden secateur or a knife. The upper cut is perpendicular to the axis and should be about 10 mm above the bud. The lower cut may be either skew or also perpendicular, located just under a bud. Graded cuttings are then tied in bundles of 50 to 100 and stored in a suitable place (either in cellars with their thicker end in wet sand or outdoors, laid in furrows). Before planting, they must be dipped in flowing water for 24 – 48 hours. This is a good method for the propagation of almost all woody species, especially poplars and willows; as to conifers, good results are obtained with this method mainly in spruce.

Propagation by summer cuttings is usually successful with English oak and durmast oak, beech and different exotic species and rare forms. The cuttings are taken in June and July from half lignified sprouts and have to be immediately planted. Their basal parts should have been dipped in growth stimulator solutions before they are planted.

Root cuttings (European aspen, grey poplar) are seldom used today. Root cuttings are taken in autumn or early in spring by cutting (under the soil surface) the horizontal roots off the parent plant. The cuttings (5 – 10 mm thick, 0.1 – 0.15 m long) are usually set horizontally (though vertical planting is also possible) in furrows 30 – 50 mm deep.

9.2.2 Afforestation

9.2.2.1 Soil preparation

The soil must have been prepared before the planting starts. The purpose of soil preparation is to improve soil and other environmental conditions, especially:

- to improve the physical, chemical and biological properties of the soil by mixing the upper humus layer with the mineral bottom layer of the soil,
- to manage the afforestation work within the short time available,
- to reduce the competition of weeds,

– to improve the ecesis of the plants and accelerate the formation of crown closure.

In principle, soil preparation may be divided into local and whole-area preparation.

Local soil preparation means removing the sod or chemically killing the weeds and cultivating the soil in the form of pockets, patches, strips, etc. The preparation may also include the digging of holes and the making of mounds, perhaps also drainage and such other technical or biological operations as may be necessary.

In whole-area preparation, the soil is treated (or broken) over the whole area.

Further, soil preparation may be either manual or mechanised.

According to the means by which the soil is prepared, the soil preparation may be mechanical, biological or chemical.

Mechanical soil preparation

The soil may be mechanically prepared either by hand or by machine. Manual soil preparation still prevails. It is very laborious, using different types of hoes, combinations of axes and hoes, and spades. The machines (implements) used to prepare the soil for planting include common farm ploughs or special ploughs, rotavators, soil cutters, harrows, smoothing harrows, mechanised hole diggers, soil breakers, etc. The following methods of mechanical soil preparation are recognised:

Pocket or patch soil preparation. The size of the patch depends on the degree of weed infestation and ranges from 0.35 × 0.35 to 1.0 × 1.0. The patch is usually a square, on slopes it may be a rectangle. The patches are distributed to form a chessboard pattern and are made by hand, using a hoe, usually in autumn. The removed sod is put on a side of the patch and the soil is loosened to a depth of 0.1 – 0.12 m. Depending on weed infestation and other microterrain conditions, 2500 to 4000 flat spots have to be made in an area of 1 ha.

Hole preparation. Holes are dug with a wide hoe, axe-hoe or spade. Their size is 0.25 × 0.25 m to 0.8 × 0.6 m, depth 0.25 to 0.30 m. For poplars the respective dimensions are 0.8 × 0.8 m and 0.6 m. The holes for poplars and for half saplings and saplings can be dug with hole diggers mounted on a tractor. For this, the terrain must be flat, free from waterlogging and the soils have to be skeletonless.

Soil preparation for mounds. This method is resorted to in heavily waterlogged and weed-infested places: the mound is made manually, its base being 0.5 × 0.7 m in size and height 0.2 – 0.5 m. The spacing between the mounds depends on the number of plants per ha, on the species, stage of development and other factors. The number of mounds per ha usually ranges between 2500 and 4500.

Mound preparation. This technique is used in waterlogged, muddy or inundated plots which cannot be drained without excessive costs. The mounds are made (using ploughs) 1 – 2 years before planting. They may be either continuous or interrupted, 0.6 – 1.0 m wide and 0.2 – 0.4 m high. At a spacing (measured from mound axis to axis) of 2.0 – 2.5 m, there will be 4000 – 5000 m^2 of mounds per ha.

Furrow soil preparation. Furrows are ploughed in weed-infested places on gentle slopes with a layer of raw humus on the surface. Furrows,

0.2−0.3 m (one-sided) or 0.4−0.7 m (two-sided) wide, are made with a plough and may be either continuous or interrupted, the spacing between the centres of the furrows being 1.0 to 1.5 m. Plants are then set or seeds are sown onto the bottom of the furrow.

Strip and belt soil preparation. The soil is cultivated to a depth of 0.05−0.15 m in strips 0.4−0.8 m wide or to a depth of 0.12−0.18 m in belts 0.7−1.5 m wide. The strips are usually made by hand and the belts are prepared mechanically, using ploughs, rotavators, disk harrows, cultivators and other implements. Again, they may be either continuous or interrupted, depending on the microrelief (forest/nonforest soil, presence of stumps, boulders, etc.).

This method of soil preparation replaces cultivation of compact areas in light-textured soils exposed to the danger of wind or water erosion.

Whole-area preparation. This technique of soil preparation involves the whole area to be planted. It can be used on nonforest soils, e.g., when establishing "green lines" in flat areas or on gentle slopes. This method is also used with advantage on soils with compacted upper layers.

Biological soil preparation

Biological preparation follows mechanical preparation, using the land-improving effect of pioneer woody species which prepare the conditions for the target species. The typical pioneer species include aspen, goat willow, European birch, alder, crane and others. These species have modest requirements for the soil, suppress weeds, improve the microclimate, are able to remove excess water from the soil, are hardy − resistant to drought, excess moisture, frost and wind − tolerate shading and high exposure to sunshine, can enrich the soil with nutrients and have other favourable properties.

Biological preparation is primarily used in the reclamation of dumps, in devastated forest land and in some cleared areas that are difficult to afforest.

Chemical soil preparation

Chemical soil preparation provides easier conditions for mechanical preparation by killing the vegetation (usually herbs) in the spots where tree seeds are to be sown or plants are to be set. Different kinds of herbicides are used for this purpose.

Chemical soil preparation is also useful on degraded soils, on high-acidity soils, on soils very poor in nutrients, etc. This type of soil preparation is based on the application of the most needed nutrients (calcium and others).

9.2.2.2 Afforestation techniques

Sowing

Sowing corresponds to the natural manner of soil regeneration and offers the following advantages: all nursery work and associated operations are avoided, the plants stay in the same environment during the whole course of their development and their root system remains intact, sowing is faster, requires simpler techniques and is cheaper. On the other hand, sowing also has a number of weaknesses, including, for example, the following: large amounts of seed are required, the soil has to be thoroughly prepared, the conditions for the growth of the young plants

are much less favourable than in the nursery, the plants are exposed to a much greater danger of damage by drought, weed competition, insects, frost, snow and browsing, the sowing often has to be repeated, etc. Hence, sowing is only advantageous in locations where the seedlings will have favourable conditions for good growth and development, e.g., in weed-free areas and at sites where the seedlings would be protected by a tall stand; however, sowing is also resorted to in extreme locations where planting would be technically impossible or extremely costly (rocks, steep gullies, very shallow and poor soils, etc.). Sowing should be avoided in high mountains, in frost hollows, on heavy-textured, compacted, dry, waterlogged and inundated soils, highly podzolised soils, soils vulnerable to drought, soils with a layer of highly acid raw humus, soils very poor in nutrients, heavily polluted soils (spoil banks and dumps), heavily weed-infested soils, etc.

Sowing can be divided into:

a) *local sowing,* which includes spot (pinch) sowing, pocket (patch, nest) seeding and row (strip) seeding,

b) *broadcasting.*

Spot (pinch) seeding. The planting of 2−3 large seeds (acorns, beechnuts, walnuts, chestnuts) at a spacing of 0.3−0.5 m, using a hoe or a planter, etc., is called spot seeding. If small seeds are sown in this way, the method is called pinch sowing, but it is however, practiced very rarely.

Pocket (patch, nest) seeding. By these methods the seeds are sown onto spots of different shapes (rectangle, square, ring, ellipse, triangle). These spots are prepared either in advance or together with the sowing. The size of the pockets is about 0.35 × 0.35 m, patches 0.5 × 0.5−0.6 m and nests 1 × 1 m. The heavier the weed infestation, the larger the dimensions of the patch to be prepared.

Row (strip) sowing. Seeds are sown into prepared furrows, strips or bands, using simple sowing machines. The preparatory (pioneer, supporting) species are sown over the whole area of the strip or band, acorns are planted in lines and the seeds of other species (e.g., European ash, Norway maple) are sown onto belts 0.1 m wide.

Broadcasting. Broadcasting is most commonly used to sow the pioneer species such as birch, alder, crane and others. The seeds of these species are very small and have to be mixed with sand at a ratio of, say, 1 : 3. This technique is used on a minimum scale because it requires soil preparation over the whole area, a large amount of seed is needed, the tending of such stands is difficult and the stands require frequent interventions. Sowing rates (in kg per ha, Tab. 82) have been determined according to the sowing technique and species of the seed.

Sowing depth. Deep-sown seeds are not able to sprout and emerge safisfactorily (they germinate irregularly or fail to germinate at all). Shallow sowing may expose the seeds to the danger of drying, though the drought may be mild and short, and loss by predators. Sowing depth depends on the species, on the soil and other factors. The generally applicable principle is that the larger the seeds the greater the sowing depth.

Planting

Compared with sowing, planting is much more frequent and much more common at all types of locations. Afforestation by planting offers the following advantages, compared with seeding: much less seed is needed, less plants are

Table 82

Seeds sown per 1 ha of area being afforested. Sowing rate.

Genus	Seeds in kg per ha			Sowing depth (cm)
	row sowing	patch sowing	broadcasting	
Oak (*Quercus* L.) Ash (*Fraxinus* L.)	250–300	200–250	–	4–8
Maple (*Acer* L.)	34–45	30–40	–	1–2
Mountain ash (*Sorbus* L.)	20–22	18–20	–	1–2
Elm (*Ulmus* L.)	20–25	15–18	–	onto soil surface
Alder (*Alnus* MILL.)	12–15	10–12	–	onto soil surface
Birch (*Betula* L.)	15–20	10–15	30–40	onto soil surface

consumed, mixed stands can be formed and spacing can be controlled, the plants have strong root systems allowing the stand to develop a canopy quickly, control weeds and resist extreme effects, the period for which the stand needs tending is shorter, etc. The shortcomings this approach has are as follows: higher costs (to grow the plants), laborious stand-establishment work, deformation and damage caused to plants during transplanting, etc.

The following types of planting are recognised:

– *according to the number of plants set* in one place (patch): single planting, double planting, bunch planting;

– *according to planting method and technique* (using naked-rooted plants):

1. deep planting, including pit, notch, furrow and corner planting (Fig. 217a–e)

2. elevated planting, including hillock and mound planting (Fig. 217f);

– *according to the type of plants with rootballs*: ball plants, plants with peat-and-pulp pots, plants in plastic bags and grown-up tree and shrub transplants.

Single Planting. One plant is set in each planting spot. The spacing may be either regular or irregular.

Double planting. Plants of two species are set at each planting spot (into a pocket or into one notch or two notches 0.05–0.15 m apart). The double-planting pairs usually combine a pioneer species with the target species.

Bunch Planting. Several plants (usually the same species) are set at a close spacing at each planting site. This method can be used with advantage in establishing special-purpose plantations, mainly in extreme locations where it is difficult for a single plant to resist the harsh conditions.

Pit planting. The commonest planting technique used under Central European conditions. It has a number of modifications. The process is typically as follows: remove the litter or sod and dig a hole 0.25 × 0.25 to 0.5 × 0.5 m in size (depending on the size of the root system, type of soil and species of the tree or shrub to be planted). Put the upper humic soil layer aside (to spread it on the roots of the plant). Put the roots in the middle of the hole, hold its trunk in one hand and spread the earth over the roots by the other. While filling the hole lift the plant

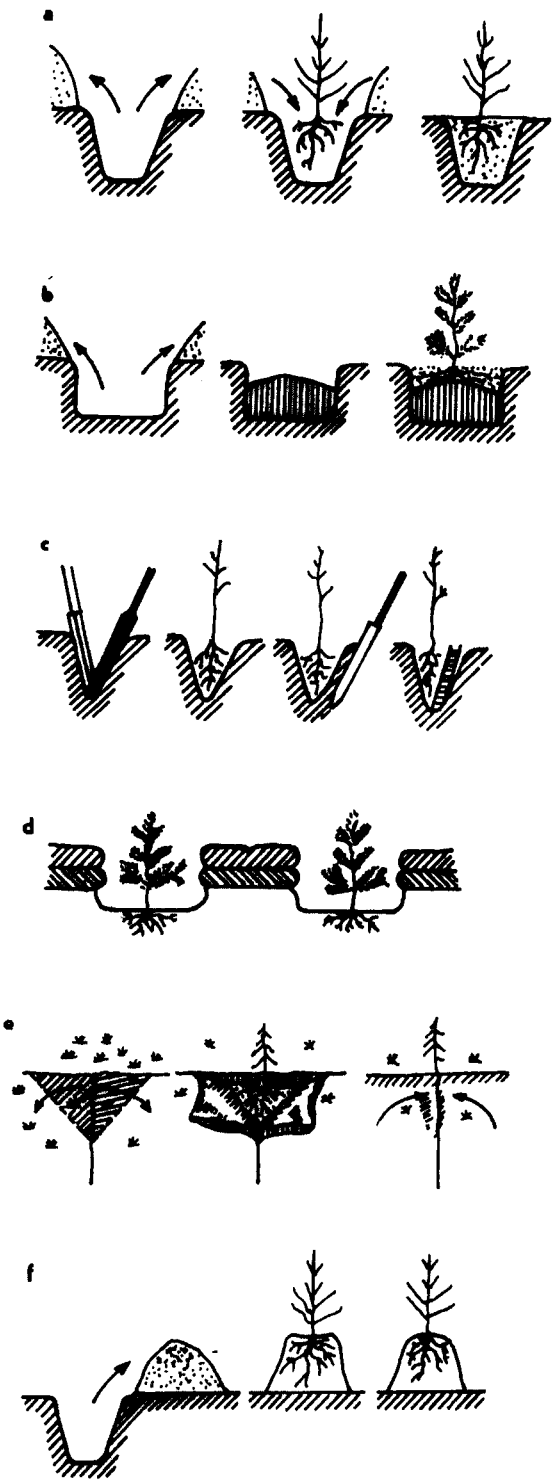


Fig.217 Planting methods:
a – pit planting – deeper root system,
b – pit planting – shallow root system,
c – notch planting,
d – furrow planting,
e – corner planting,
f – mound planting.

slightly to improve the contact of all rootlets with the earth. Compact the soil lightly during filling. When the filling is completed compact the surface layer more closely with the foot.

Pit planting is applicable in all soil types and can be used with one-year old plants as well as those in the 4- to 5-year category and, of course, with saplings. If the method is performed carefully, the roots are not deformed. The plants are usually set 10 to 20 mm deeper than they grew in the nursery.

Notch planting. The second most common method after pit planting. Before the notch is made, weeds, litter or humus (if it forms a thick layer) must have been removed, using a hoe. Then the planting spade is driven into the soil and shaken to and fro to open a notch of the required size (e.g., 0.1 m wide and 0.2 to 0.3 m deep). Another worker then puts the plant to a certain depth into the notch, lifts it a little and uses a small trowel to spread the roots, taking care not to allow them to bend. Then another one or two notches are made near to the first notch with the plant. The other notch(es) is left open because it will gather water and humus for the plant: it should only be closed if the planting is performed in autumn because frosts might damage the plant's roots.

Notching is used with advantage to set weaker plants whose root system will fit well into the narrow notch. It is suitable in light-textured sands to sandy loams and unsuitable in heavy and compacted soils with a tendency to drying and in plants with an amply branched root system because the narrow notch would deform the roots.

Furrow planting. The plants are set by and in various ways or by a suitable mechanised method into furrows prepared in advance.

Corner planting. A T-cut is made in the sod using, for example, an axe-hoe, the corners are unwrapped and the naked roots of the plant are slipped underneath. The sod is then stepped on to hold faster. This method can only be practiced in areas with sufficient rainfall and in plants with a flat root system, e.g., spruce, and in areas not heavily overgrown with weeds.

Hillock and mound planting. These two planting methods are used in waterlogged, inundated and weed-infested soils. A hole is dug in the place where a hillock was made or a mound was ploughed and the plant is put into the hole. The plant's roots should reach into the original soil underneath. When planting pioneer plants, especially by the mound method, the plants may also be set into notches.

Ball planting. This method removes all disadvantages of all methods in which naked-root plants are used. The plants are put into the soil with a ball of earth in which they have grown. There is minimum danger that such plants would dry out or be damaged by careless handling. The plants may be obtained from self-seeding but may also be prepared in nurseries or seedling enclosures. Spades, either ordinary or hollow, are used to lift such plants (with the ordinary spade the plant has to be dug from all the four sides). With the technique used, ball planting is very costly and is mainly used for the afforestation of extreme locations and also when grown-up trees and shrubs are planted.

Setting plants grown in peat-and-pulp pots or plastic bags. Plants and seedlings are grown in increasing amounts in various types of containers. One- to two-year old seedlings are planted in the peat-and-pulp pots, filled

with soil, and are left in a partly shaded place. They have to be irrigated as necessary. A year later they are set together with the containers into the prepared holes. Similar techniques are used with other methods of the production of plants (in plastic containers). These methods of afforestation are used mainly in:

– afforestation of shallow, gravelly, stony soils, in soils poor in minerals and at sites with otherwise extreme conditions,

– afforestation outside the spring and autumn seasons.

Transplanting mature trees and shrubs. This is a special case of planting with a ball of original earth. To hold the ball on the roots it has to be protected with a special jute, wooden or other wrapping or container; it is advantageous to transplant grown-up trees in winter when the soil is frozen on the roots. A crane truck is used to lift such a tree or shrub from the ground and load it onto a platform trailer. After planting in prepared holes, the aerial part of the tree has to be reduced and the plants have to be anchored and intensively irrigated if necessary. The costs of moving mature trees are high, so the method is only used to extend greenery in cities, industrial areas and other places where such costs would be justified.

The planting season depends on the local conditions of the site, on altitude above sea level, on the species of the tree or shrub planted and on the type of the plants. It is advisable to do the planting at low temperatures between 7 and 10°C when evaporation and transpiration are low. For these reasons, naked-root plants are set in spring or in autumn.

Spring planting means planting early in spring when the soil surface has dried and the optimum planting time lasts until the plants start budding. To extend the planting season, the plants must have been adequately stored (air-conditioned stores, snow caches, etc.). Conifers are the best plant category to plant in spring.

Autumn planting is suitable for deciduous species, including larch. The trees or shrubs are transplanted as soon as they have shed leaves. The advantage is that the plant has time to take root in autumn and to immediately start growing in spring. The method is not suitable for conifers because the plants may be damaged by drought and frost.

Plants with rootballs can be set practically throughout the year, except perhaps in periods when the soil is frozen.

The spacing of the plants after setting forms a geometrical pattern. It may be either regular (rectangle, square, triangle and others) or irregular.

The planting density is defined as the number of plants per ha. A regular spacing is advantageous because it simplified tending and other operations. Nevertheless, the spacings are very often irregular, allowing one to choose the best place for each plant (spots with more moisture, spots protected by stumps and shrubs, places with deeper soil, etc.).

Determination of the best spacing and thereby the optimum consumption of seedlings per unit area depends on the species, on seedling quality, on the ecotope, climatic conditions, possibilities of using mechanisation, etc. Generally, the spacing should be as dense as to allow the stand to quickly form a crown closure and as economical as to avoid any wastage of planting material. The number of plants per ha and some recommended spacings are shown in Tab. 83.

Composition of forest stands and approaches to mixing the species. The majority of forest stands in the Czech and Slovak Republics consist of mixtures of several

Table 83

Number of naked-root seedlings per 1 ha of area being afforested*

Species	Area**	Thousands of seedlings	Some recommended spacings (m)
Picea excelsa (LAMK.) LINK	II, III, IV, V Ib, VIb Ia, VIa	5.0–7.0 4.0–5.0 2.5–4.5	1.4 × 1.4; 2 × 1; 5 × 1 2 × 1; 1.5 × 1.5 2 × 2; 2.5 × 1.5
Abies alba MILL.	–	6.0–10.0	1 × 1; 2 × 0.8
Pinus silvestris L.	III, V*** II, IV	12.0–20.0 10.0–15.0	1.2–1.5; 0.4 1.5; 0.5–0.7
Larix decidua MILL.	–	3.0–5.0	1.8 × 1.8; 1.5 × 1.5
Psudotsuga taxifolia BRIT.	–	3.0–5.0	1.8 × 1.8; 1.5 × 1.5
Abies grandis LINDL.	–	3.0–5.0	1.8 × 1.8; 1.5 × 1.5
Pinus strobus L.	–	6.0–8.0	1.5; 0.5–0.6
Pinus mugo TURRA	–	2.5–4.5	2 × 2; 1.5 × 1.5
Quercus robur L. *Q. petraea* MATTUSCH	–	12.0–17.0	1.25–1.5; 0.5–0.6
Tilia L., *Acer* L., *Fraxinus* L., *Carpinus betulus* L., *Quercus rubra* L.	–	5.0–10.0	1 × 1; 1.5 × 0.8
Betula L. *Populus tremula* L.	–	5.0–8.0	1.5 × 1.5; 2 × 1.2
Alnus glutinosa L.	–	3.0–5.0	1.8 × 1.8; 1.2 × 1.2
Tree willows *Salix* L.	compact planting row planting	2.5 0.5 km	2.0 × 2.0
Populus L.	compact planting row planting	0.400–0.625 0.333–0.250	4 × 4; 5 × 5 3 × 4; 4 × 4

* Data on related species (possibly with adjustment) are used for species not shown in the table.

** Areas III, V, VIII, IX – lowlands and hilly regions (below 165),
II, IV, VII, X – noncompact medium-altitude mountains (130–165),
I, VI – mountain massifs (below 130), Ia, VIa – high mountains (below 100) – in parentheses: adequate growing season, days

*** The upper boundary for the Scotch pine applies to one-year old plants (1/0). For two-year old seedlings (2/0) or one-year old seedlings left in the nursery for another year (1/1), this upper boundary is reduced to 18 000 plants per ha.

species, some having only a temporary mission. Single-species stands (monocultures) are justified only in some places with extreme conditions where other species' self-seeding is expected, in projects involving the use of pioneer plants

and in cultures of plantation type. Mixed stands offer the following advantages: a better utilisation of the conditions of the site (nutrients, light, sunshine), larger volumes of wood produced, improvement of soil fertility, better resistance to the harmful effects and generally better performance of all forest functions.

The following groups of species occur in the mixed stands:

a) main species – largest proportion,

b) admixed species – above 10%,

c) casual species – below 10%,

d) soil improving species – 10–15% (these species improve the soil by the leaves they shed or by assimilating atmospheric nitrogen, e.g., alder, locust).

Casual and admixed species improve the quality of the stand, protect the soil, force the main species to grow faster and do not allow suckers and sprouts to overgrow the tree trunks.

The following basic methods of mixing the species can be used to establish mixed stans:

a) distribution of individual trees – used only when ameliorating species are added,

b) distribution by rows – advantageous when line-type stands are established (riparian stands, windbreaks),

c) distribution by groups – the most frequent approach, best for the environment.

The locations where separate groups of species are planted can have different shapes (circle, ellipse, square, irregular shape, etc.). The smallest group should not be smaller than the ground plan of the crown of a single tree at cutting age, i.e., the area occupied by the group should not be less than about 25 m^2. Excessive variety in the species spectrum and the use of species severely competing with each other are not recommended.

9.2.3 Treatment and protection of cultures

During the first years of life, plants (seedlings) are exposed to various unfavourable effects which may damage or even destroy the stand, however carefully it may have been established. Hence, once planted, the cultures must be protected against pests and other damaging factors, both biotic and abiotic. Suitable tending measures also have to be taken to support the growth of the plants.

Loosening. The soil should be loosened to a depth of 30–50 mm (damage to the roots of the plants must be avoided). The time to perform the loosening operation and its frequency depend on the soil conditions (heavier and drying soils have to be loosened more frequently), on precipitation and on the occurrence of weeds. Loosening has to be performed during the first and second year after afforestation. The best time for it is late spring when the plants grow most intensively.

Weed control. In the newly afforested areas, up to 90% of the weed plants must be killed. This may be done by a biological, mechanical or chemical method. The biological methods include adequate composition of the culture and also mulching (putting the mown weeds around the plants). The most frequent mechanical intervention is weed mowing. The first mowing should be performed just before the

weeds start flowering. Sun-loving woody species are especially vulnerable to shade. Species that prefer shade should not be abruptly exposed to sunshine, so the weed stubble left among the plants should remain taller. Cultures endangered by the lodging of the weeds (in winter) must be treated before the onset of winter. To save costs and time, weeds may also be controlled by trampling but this method is not very effective. Chemical weed control is based on the application of herbicides; this is an inexpensive and fast method, but may damage the environment.

Young cultures are sometimes damaged by suckers and shoots and, at more advanced stages, also by the auxiliary species which have to be controlled either mechanically (cutting) or chemically (use of arboricides).

Much attention has to be paid to the biotic damaging factors: browsing and fraying. The newly planted cultures have to be fenced, an effective but costly measure. Repellents may sometimes be used to prevent browsing. Mechanical measures include the covering of the terminal shoots with glass wool or other (solid) materials.

9.2.4 Filling the gaps and improving the stands

Losses of the young plants are usually suffered in the newly planted areas: the plants may be killed by drought or by game, or may fail to compete with the weeds. The extent of the losses is determined during the regular autumn inspection.

The gaps resulting from the dead plants have to be filled and the culture must be improved if

a) the total loss is in excess of 10% at a stand density of more than 5000 plants per ha,

b) the total loss is above 50% with less than 5000 plants per ha,

c) the total loss is less than under a) and b) but the gaps are concentrated in certain places, not evenly distributed over the whole area.

The gap-filling (improving) intervention should be performed as soon as possible to allow the newly planted (sown) additions to catch up in growth with the original stand. Smaller gaps need not be filled (self-seeding can be relied on). The plants to be used to fill the gaps should be taller and more advanced; ball plants are also a good solution. Fast-growing species, advanced plants or shade-demanding crops have to be used to fill the gap if the stand around the gap is taller (the shade-demanding species would form the understorey). The species used for filling the gap should match with the original intention of the forest management.

9.3 Tending of stands

Tending includes a set of practices and interventions to support the forest associations and to provide conditions for achieving the targets of management as soon as possible. Tending is based on systematic and targeted selection, using the following criteria:

1. sanitary criterion: remove the damaged and dead individuals from the stand,
2. quality criterion: remove individuals of poorer quality to improve, by each

intervention, the general quality of the stand. Quality selection is effected in two ways:

a) negative selection: remove the undesirable individuals (those of poor quality, those of nonstandard shape or undesirable species),

b) positive selection: identify the best individuals and remove those that would hinder their development. This approach improves the quality of the stand directly, so it is more important,

3. species criterion: species of lower value (e.g., the pioneering ones) are removed or the target species are identified and supported by suppressing the growth around them.

The tending operations can be performed at different tree height levels, including:

a) high (crown) operations – in the upper tree layer,

b) low operations (operations from below) – in the lower layers of the trees.

The following tending operations are distinguished (by tree age while the trees are young, later by diameter):

- hand thinning, pulling and clipping in new growths and young crops (before they are 2 m tall),
- regular cleaning and moderate cleaning in thickets and in stands of small pole stage (2 to 8 m tall),
- thinning in stands in pole stage (stands from a height of 8 m to felling maturity),
- heavy thinning to free the trees in high forests, starting from chest height diameter of 0.23 (chest height: 1.3 m) (in systematically tended stands at medium to good quality sites),
- pruning (additional tending operation to improve the quality of the stand).

9.3.1 Techniques of forest stand tending

Tending techniques are operations representing the silvicultural treatments of stands. Depending on stand age and development, they include:

- tending of new growths and young crops
- tending of thickets
- tending of stands in small-pole and pole stages
- tending of maturing stands
- silvicultural treatment of maturing stands.

9.3.1.1 Tending of new growths and young crops

Self-seeded growth are young seedlings in a stand which is not yet secure. New growth is a young self-seeded stand which is already secure. Young crop is a stand which originated from planting or seeding and has not yet formed a crown closure.

The tending interventions in new growths and young crops include pulling and clipping to control the overtopping trees, to check root suckers and stump sprouts, remove individuals of poor health and nonstandard form, individuals damaged by browsing, frost, logging operations, etc., to develop economically and environmentally suitable mixtures and reduce the number of excess trees.

Pulling is performed at 3- to 5-year intervals (as required) to thin the new growth of species having taproots. Species whose roots are shallow and spread over a wide

area must be clipped because pulling would damage the roots of the trees around. Excess individuals from self-seeded young crops can be lifted for transplanting, using a hollow spade.

Special attention has to be paid to the overtopping trees which usually are unshapely individuals with a large crown at the top layer of the stand. They should be removed during the first intervention unless they have the mission of protecting the new growth against direct exposure to sun, frost, snow and wind. Overtopping trees of sun-preferring species such as larch, alder and others may be an exception.

In well-tended young crops, the trees may also be shaped by pruning. However, pruning is costly and its effect does not last long. In hardwood species of the highest commercial value it is sometimes useful to shape the crown (if the shape was distorted by external effects). In conifers the shaping of the crowns is not a usual practice: the only thing that can be done is to remove bifurcated trunks (e.g., in the Douglas fir, silver fir and Norway spruce).

9.3.1.2 Tending of thickets

Once new growths and young crops form a crown closure, the trees begin to compete but natural selection through competition cannot guarantee that the management targets will be met. Hence, artificial selection is also needed.

Thicket is a young forest stand at an age of about 10 to 20 years whose height is more than 1 m and whose diameter at the height of 1.3 m is less than 70 mm. Mild cleaning is used today in the tending of thickets to remove all obviously poor and excessive individuals. Mild cleaning includes silvicultural treatments during which individual trees are removed from the upper layer in several repeated operations following each other at fairly short intervals. The individuals that are removed include diseased, damaged or poorly shaped trees (crooked, forked or irregularly branched trees, excessively branched overtoppers, etc.) and trees of non-target species. This provides better conditions for commercially and biologically important species whose growth has been suppressed. The intensity of the intervention must be less than in regular cleaning: the stand must remain a thicket after the treatment. Under such circumstances, the treatment must be repeated more frequently (e.g., every 2 – 4 years).

In mild cleaning operations much attention must be paid to mixed stands where sufficient space must also be provided for the slow-growing target species. Coniferous overtopping trees usually do not do much harm because they do not grow much in width. Careful tending of thickets also includes pruning which, however, is done only infrequently.

9.3.1.3 Tending of stands in small-pole and pole stages

A small-pole stand is one in which the trees' breast-height diameter is greater than 70 mm (21 – 40 years). Stands in pole stage are older (41 – 60 years), the diameter of the trees being up to 0.25 m. The silvicultural treatment of stands in these two stages is based on thinning: commercially promising individuals are selected in the stand and their growth is encouraged by the removal of less promising trees obstructing the growth of the selected ones. The intensity of thinning must not be too high: the treatment should not substantially reduce the

stand density and should be controlled according to the type of the site, species of crops grown and the state of the stand.

Depending on the layer of the stand in which the treatment is performed, thinning may be:

- low (thinning from below)
- high (crown thinning)
- combined.

Depending on their intensity, five degrees of thinning are distinguished, ranging from light thinning up to heavy cleaning. The intensity of the thinning depends on the size of the trees, on the formation of the trunks and crowns, on the proportion of species of commercial value in the upper layer, and on all characteristics of the ecotope.

Pruning is another operation used in the tending of stands in the small-pole and pole stages. The purpose of the pruning treatment is to develop good-quality, smooth and healthy trunks, to prevent the occurrence of wood-decaying fungi, to encourage length growth (especially in conifers), etc. Both dry (dead) and green branches can be lopped.

9.3.1.4 Silvicultural treatment of maturing stands

Maturing stands, called trunk timber or stem wood, whose growth in elongation has culminated and whose trunks already have a full commercial value (age of about 60 years and older) are subjected to further thinning to reduce stand density: more space is provided for the best trees to encourage increment of their diameter. Of course, thinning at this stage cannot be very intensive in extremely sun-exposed locations.

Besides its commercial objectives, the tending treatment of maturing stands should also already be aimed at preparing conditions for future regeneration of the forest. Having more space around themselves, selected trees are capable of producing larger quantities of good-quality seeds both for seed harvesting and for self seeding.

9.4 Soil-conservation forest stands

There is much forest and non-forest land under Central European conditions where vegetation stands need to be established to protect the soil: areas exposed to sheet erosion and wind erosion, landslide areas, steep slopes with low soil stability, areas covered with debris, cleared areas of various types, etc. In recent years, this category has been extended by areas such as dumps, spoil banks, vast areas affected by air pollution, etc.

The greatest obstacles to afforestation efforts in such areas include the high degree of nonstability of the soil and often the absence of topsoil (the parent rock is often seen on the surface of such land). Such areas with minimum vegetation are exposed to strong degradation (if not devastation) processes (formation of scours and gullies). The intensity of these processes increases with the deterioration of the soil conditions (upper layer of the soil removed), microclimatic conditions (the soil dries out and its upper layer is exposed to wider temperature extremes) and vegetation conditions (vegetation cover diminishes and loses quality).

Under such conditions special approaches to vegetation management have to be used. Generally, before vegetation is established the soil conditions may have been adjusted by changing the microrelief (terracing), by mechanical binding (using technical means), by drainage, and by biological treatment (sodding, seeding of different plants, and afforestation itself). The most common approach is to combine the different soil-protection measures.

9.4.1 Grassing

Some slopes that cannot be afforested for any reason can be protected by grassing. Today, the most efficient method of grassing is "hydroseeding", using water as carrier medium for the seeds (usually with the addition of fertiliser, peat and binding agent). If all components are used in an appropriate ratio, the method is reliable and efficient, particularly at sites exposed to water and wind erosion. Several hydroseeding technologies have been developed, using different machines and different anti-erosion substances (which protect the slope before the grass develops) – e.g., Bitusprit, Breht, Finn, Wego-Sect, Thanner, BASF, Hydrosa-Verdyol, Agrosil.

Special spraying equipment, called hydroseeders (e.g., the Riedberg, Fin-Seeder), is used for this process. The machine is either mounted on the chassis of an off-the-road vehicle or on a trailer and operated from the towing vehicle. It has a tank holding 2 to 5 thousand litres (with a mixing device), a water gun and a powerful self-priming pump. The seed mix is continuously agitated inside the tank to provide uniform seeding. Depending on the type of machine used, the effective range of the equipment is up to 40 – 50 m.

Before the actual sowing, the soil surface must have been well smoothed and must not be chemically treated; irrigation must be provided if the weather is dry. The best seeding periods are April to June and September to October. The months of July and August are less suitable because in the driest summer period it is necessary to irrigate the area 2 to 3 days after seeding. Sowing should start from the top of the slope and continue downwards.

The seeding rate, including the amounts of all components of the hydroseeding mix, is as follows (per 100 m^2):

2 – 4 kg of mixed grass seeds
12 – 25 kg of peat
10 – 15 kg of pulp (as binding agent)
8 – 11 kg of commercial NPK fertiliser
100 – 150 litres of water.

The composition of the grass-seed mixtures is based on the conditions of the site, on the climatic conditions, on the purpose of grassing and on the biological properties of the individual grass species. The mixture usually consists of 3 to 5 species, 40 – 50% of the total amount being represented by the seeds of bushy grasses and 50 – 60% by the seeds of stoloniferous grasses. The suppliers of the hydroseeding mix are able to provide their product to fit different locations, e.g., seed mixtures for dry or humid sites, etc., "technical mixtures" for slopes, irrigation and drainage canals, etc.

To adequately protect slopes against erosion (before the grass stand is fully established) it is recommended to spray the sown areas with the protective emulsion of Antieróza (it is not recommended to apply the emulsion with the seeds because it reduces their germinability). Antieróza is a white emulsion based on synthetic latex and oil. Before use it is diluted with water. After spraying, the emulsion penetrates into the upper layer of the soil and develops into a filmy net; it forms a grid linkage among the soil particles and maintains its effect for about 8 months. The Antieróza film is permeable to water but is able to prevent excessive drying of the soil. It protects the sown areas and also the young plants when they emerge. The following principles have to be respected when working with Antieróza:

- the water for its dilution must be chemically clean and must not contain too much chlorine and other salts (maximum hardness 25°) (German scale),
- the spray should not be applied onto frozen soil or at temperatures below 5°C,
- the application rate is about 1 litre of solution per m^2.

On steep slopes and sunlit slopes the effect of the application of the binding material can be supported by mulching. The mulching material provides temporary protection of the soil against erosion, provides shade for the seeds on the slope and keeps more moisture in the soil. Peat, pulp, chopped straw, hay and other materials can normally be used for mulching. If peat and pulp are used, they ought to be sprayed before Antieróza treatment. If straw is used, the grass-seed mixture must have been sown by hand onto the slope, together with an addition of commercial fertilisers, a 30 – 50 cm layer of chopped straw is then spread on the surface and a binding material (Antieróza, sulphite bitumen) is sprayed on top.

9.4.2 Soil-conservation afforestation

The establishment of forest stands in degraded (up to devastated) areas is among the most difficult and laborious afforestation operations and has to be preceded by thorough field investigations. To prepare an afforestation project it is necessary to know the soil conditions (soil class and type, soil depth, physical and chemical properties of the soil and the parent rock), the climatic conditions (particularly the annual curves of precipitation and temperature), the vegetation conditions (species composition and structure – thickness – of the existing vegetation cover) and morphological conditions (exposure, gradient of slope, altitude above sea level, etc.).

All these characteristics, as well as the management goal, have to be analysed and interpreted to identify the best method of soil preparation, the best species structure, the intended method of tending and other operations.

9.4.2.1 Soil preparation

The more severe the site conditions, the greater the importance of soil preparation. Of course, species more fastidious about the environmental conditions will also require better preparation of the soil. In devasted areas, the generally required soil preparation operations should be combined with other measures, e.g., to prevent water or wind from removing the soil, to prevent water from washing away the seeds or scour under the roots of the young plants, to prevent mud from suffocating the plants and generally to provide good conditions for the afforesta-

tion work. This can be done by a number of biotechnical and technological measures.

Terracing

In afforesting of sloping terrains, terracing is among the simplest methods of improving the stability of the soil and weathered rocks. This technical approach reduces the gradient of the slope (adjustment of the microrelief) and divides it into a number of shorter segments, thus substantially reducing surface runoff, hence also erosion. In general, the following types of terraces are distinguished: earth (trench) terraces, fence terraces and rock terraces (Fig. 218).

Earth terraces can be effective on slopes up to a gradient of 25°. If they are built on steeper slopes they fail sooner than the trees or shrubs are able to

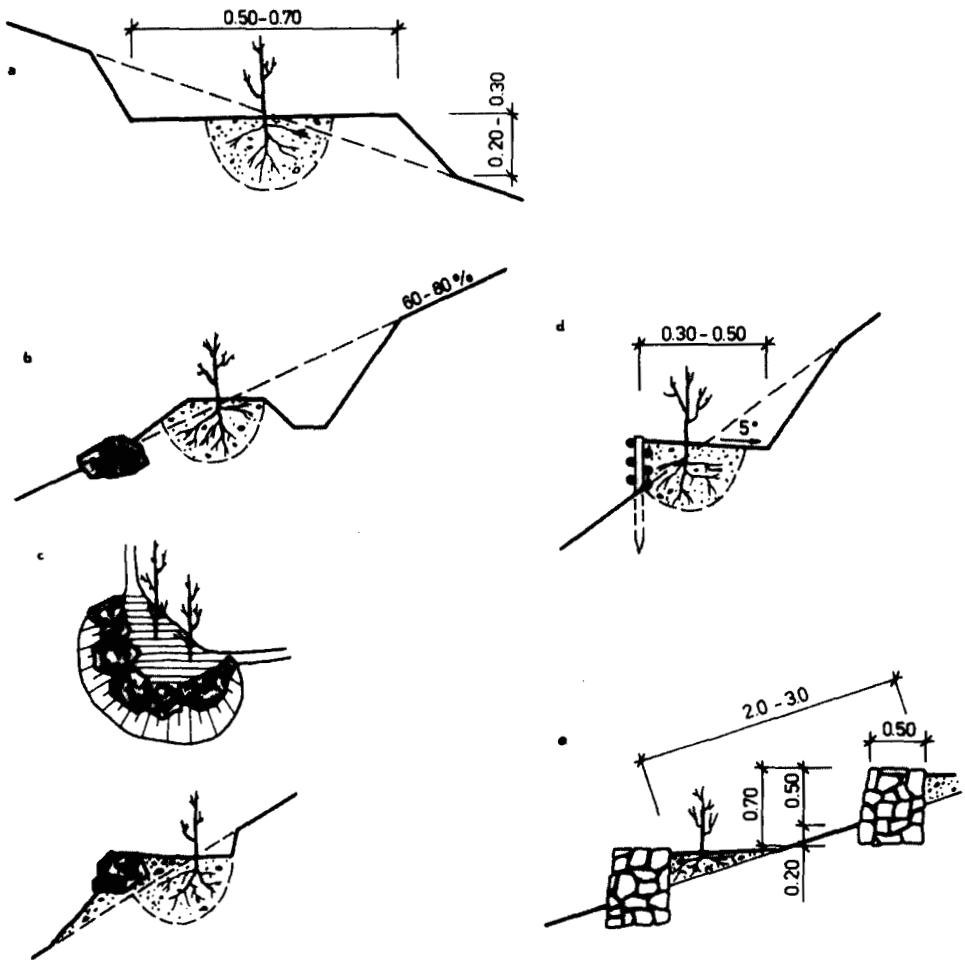


Fig.218 Types of terraces: a – earth terrace, b – earth terrace with channel, c – channel terrace (view, section), d – fence terrace, e – rock terrace.

permanently bind the soil. The earth terraces are usually 0.3 to 0.5 m wide and 0.7 to 1.0 m long. Shallow trenches about 0.5 m long may lead to the terrace from both sides to gather surface water (trench terraces).

Terraces should be built along the contour lines of the slope and should form a checkerboard pattern. It is important that they should have been built a year before the actual afforestation to let the original earth and the new bank to match each other's water regimes. The slopes on which earth terraces were built are usually afforested by planting. It is a disadvantage of terraces that earth progressively falls from above into the terrace's excavation and buries the plants (and especially the seedlings) underneath: this worsens the plants' rooting and deteriorates the conditions for the growth of the newly formed crop. To avoid this, the terraces may be built wider so that the plants can be set farther from the excavated part of the terrace.

Fence terraces are built on steeper slopes and on noncohesive soils. If an appropriate technology is used they have a greater stability than earth terraces and may remain effective for about 10 years. Built along the contour lines, they are 1.5−4.0 m apart and their width is 0.3−0.5 m. The fences consist of stakes about 60 mm thick and 0.6−0.7 m long, driven into the ground at a spacing of 0.5−0.8 m. Brushwood (30−50 mm thick) of oak, hornbeam, willow or other species is woven between the stakes to a height of 0.30−0.35 m. It is important that the terraces should be slightly inclined (about 5°) against the slope. The plants should be set on the boundary between the terrace's excavation and bank. The soil must have been well dug to provide a good seedbed or good soil environment for the plant: this will improve the catching of the seeds or plants.

Rock terraces are rarely seen because they are costly and labour intensive. They may be built on devastated slopes covered with rocks and almost devoid of soil. Rock terraces have a wall built along the contour lines from rocks collected at the site. The space behind the wall is filled with earth which is either dug between the terraces or brought from elsewhere. Terrace steps built in this way are then afforested.

Preparing the soil by building berms

Berms ("paths" or ledges similar to those built for fence terraces) are dug in this method of soil preparation but the wicker fences protect the terrace on its excavated side, not on its bank side (Fig. 219). The space between the fences and the excavation slope is filled with material from the berm above. The advantage is that the plants set on the berm are protected by the fence which does not allow any earth to fall down and bury them. The fence improves the microclimate, especially when the brushwood that forms the fence is left to vegetate. This method can be

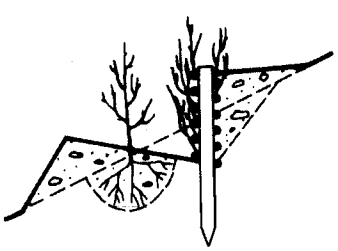


Fig.219 Preparing the soil by building berms.

used with advantage on damp soils of poor cohesion on which the plants themselves take on the soil protection (stabilising) function after the fence disintegrates.

Preparing the soil by the formation of cordons

Cordon planting (Fig. 220) is used when soil protection stands are built on steep slopes. Cordons are cuts dug in the slope along the contour lines 1 – 3 m apart. The bottom of the cuts is filled with a layer of fresh (living) brushwood or willow cuttings, sometimes even rooted cuttings. The building of the cordons starts from below: when the first cut (about 0.5 m wide) is dug, it is filled with the fresh brushwood or cuttings which are then filled with the material from the next cut dug above the first.

On steep slopes or in landslide areas the stability of the cordons can be improved by laying two long poles on the bottom of the cut (excavation); then a layer of brushwood (may include conifers) is put onto them and a 0.1 m layer of soil is spread on top (Fig. 220). Willow cuttings are then laid 20 – 30 mm apart on the bed prepared in this way and are covered with the earth from the next cordon cut.

Wicker fences

Wicker fences (Fig. 221) are used in areas endangered by sheet erosion or surface landslides and may also be erected to stabilise shallow cuts and gullies or fine debris. They are built in lines 1.0 – 1.5 m apart and are 0.1 – 0.2 m high (above the surface): they should also reach 0.1 to 0.15 m under the ground to prevent surface water from scouring under them. Wooden stakes 60 – 80 mm in diameter

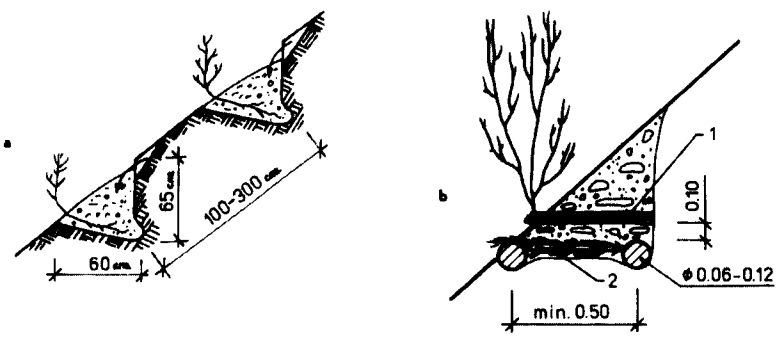


Fig.220 Preparing the soil by the formation of cordons.

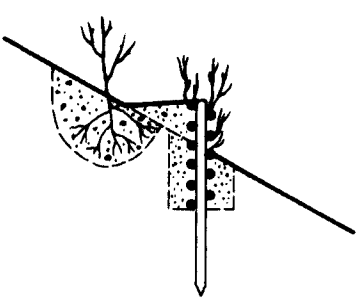


Fig.221 Wicker fences.

(120 mm at the maximum) and 0.5 − 0.7 m long (1.2 m at the maximum) are used to build the fences (their dimensions depend on the looseness and amount of material they control). The stakes are driven into the ground 0.6 − 0.8 m apart. The material between the stakes may either be fresh and alive (wickers, willow brushwood, to develop into vegetated fences) or dead (brushwood of alder, hazel, hornbeam, beech spruce, etc.).

An important role is played by the distribution of the wicker fences which depends on the extent of damage to the soil. The fences may form either solid or broken lines. Broken lines of fences, distributed to form a checkerboard pattern with overlapping ends of the fences in the neighbouring lines, are less costly to build, as compared with the solid lines. However, the best soil protection effect is achieved when the fences are built in solid lines running aslant over the slope (angle of 45°), the area between the fences being further protected by sown low vegetation.

The space behind the fences is filled with soil taken from the area between the fences or, in extreme cases, brought to the site from elsewhere: this in fact forms terraces onto which the plants are set.

Lath fences may be erected instead of the wicker fences. To check the pressure to which such fences can be exposed, some stabilisation is needed, using stakes, in extreme cases also ropes.

Clayonnage

Clayonnage is the covering of the slope surface with the brushwood of different woody species. It is used to protect hillsides not yet devastated (no gullies have developed) though no longer having their protective cover which would control intensive erosion. Clayonnage may also be used to protect sites where much debris is concentrated. The brushwood is fastened to the ground with poles or wire (2 − 3 mm in diameter) fixed to stakes driven into the ground. The brushwood fixes the soil and protects it against unfavourable microclimatic conditions and against moving debris, enriches the soil with organics, increases soil-moisture content, etc.

It is recommended to cover the soil surface while carrying out the afforestation. Advanced plants, preferably with rootballs, can be used with advantage. Addition of good-quality topsoil to the holes may be required in extreme cases.

Vegetating palisade wall

Palisades may be built in gully headcuts or in shallow furrows to effectively control erosion on light-textured soils. A palisade is a strong fence of stakes driven into the ground close to each other. The underground part of the stakes should represent about a third of their length. The upper end of each stake is fastened with a wire to a cross pole which is keyed into the side walls of the furrow. The wall built in this way is then immediately embedded in earth to allow the stakes to take root easily.

Vegetated walls

Vegetated walls of stone packing are often used to protect the toe of slopes exposed to erosion or may be built straight on the slope to prevent extensive

landslides in places where cavings occur. If the wall has to be tall, it is recommended to vegetate it.

To build a vegetated wall of stone packing, stakes are driven into the ground and stone and gravel are spread between them in layers, each layer being covered with osiers mixed with earth.

Vegetated walls such as those of logs or prefabricated concrete (as in torrent damming) may be built tall and strong. Fresh willow brushwood is laid over the whole width of the wall in the direction of the cross ties; instead of the brushwood, plants of woody species may be put on the wall and covered with earth.

Use of geotextile in soil conservation

Slopes exposed to erosion can be effectively protected using light-weight (up to 300 g per m^2) protective geotextiles, laid in strips in the direction of the contour lines and fixed to the ground with stakes (Fig. 222). The geotextile may be either degradable or semidegradable. If an area protected in this way is afforested, the geotextile also controls weeds. Where erosion furrows have already developed, it is better to use nonwoven (needled) textiles made of synthetic fibre, resistant to degradation. Medium heavy (up to 800 g per m^2) to heavy (above 900 g per m^2) textiles are used with advantage in such cases.

Geotextile can also be used to protect the soil surface in gullies, ski slopes, etc. Their use helps to minimise the need for earth work, reduce the volume of material transported, etc. Such materials do not disturb the appearance of the landscape and progressively merge into the natural environment.

9.4.2.2 Choosing the species and the method of soil conservation afforestation

The choice of woody species for the afforestation of devastated areas depends on the local conditions, including the soil and climatic conditions, exposure, altitude above sea level and the purpose of afforestation, taking into account also the requirements of the different woody species. Devastation can be caused by different factors which must also be taken into account (e.g., air pollution) when choosing the best species. Once formed, the young crops have to start to accomplish their mission as soon as possible: they must cover the soil surface, fix the remnants of the soil (i.e., prevent further devastation), and progressively improve the conditions of the site (enrich the soil with humus and nutrients, improve the microclimate, the moisture conditions, etc.).

The selection of the method of soil preparation and the selection of the species to be used will depend on the specific characteristics of the extreme part of the site to be afforested. Mechanised soil-preparation methods or methods requiring less money and effort are preferred everywhere possible. However, sometimes even the most laborious methods have to be used.

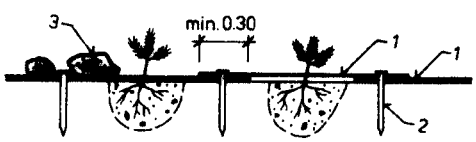


Fig.222 Using geotextiles for soil conservation.

Afforestation of devastated soils on dolomitic limestones

On gentle slopes covered with a layer of soil, the furrow method of soil preparation can be used. Holes are dug for the plants in areas where terraces do not have to be built.

Today, such sites are usually afforested directly with the target species including, first of all, Scotch pine which should form the skeleton of the future stands on the bottom parts of slopes, on slopes facing west and in places with traces of good soil, and Austrian pine on slopes facing south, at rocky sites, on mountain ridges, etc. It is often advantageous to mix these two pines together. Pines should be combined with other species, depending on the conditions of each particular site: mountain ash (*Sorbus torminalis* and *S. aria*), hazel, flowering ash *(Fraxinus ornus)* and others. As to deciduous species more fastidious about the site conditions, such stands may also include small-leaved linden, beech and hornbeam, in valleys olive ash *(Fraxinus excelsior)*, at wind-exposed sites European larch and others.

Afforestation of devastated soils on limestone substrates

Limestone substrates are much harder than dolomitic, so it is more difficult for the roots to penetrate into the subsoil. The following methods are used to prepare the soil for afforestation: notch planting with an addition of soil, pit planting with addition of soil, notch-and-mount planting, step terracing, and also making pits with an explosive, usually with an addition of soil to the pits. The explosion also breaks the parent rock to open the way for the roots to penetrate to a greater depth. Explosives save labour; about 1000 holes at a 3×3 m spacing have to be made in this way per hectare.

The pits are planted with the Scotch and Austrian pines and the following species can be added: small-leaved linden, silver linden (*Tilia argentina*), whitebeam tree *(Sorbus aria)*, hornbeam, cornel, Turkey oak *(Quercus cerris)*, English oak, common juniper and others.

Afforestation of low-productivity sandy soils

Measures can be taken before afforestation to improve the fertility of these soils. Fertilisers may also be applied.

Scotch pine is the main species to grow on sandy soils. As to the suitable deciduous trees, English oak and durmast oak and the small-leaved linden are the target species. The ameliorating species for sandy locations include European birch, the red oak (*Quercus rubra*), alder, hornbeam, sycamore maple; in arid and warm areas sycamore maple, locust, Mahaleb cherry (and others) may also be suitable.

Afforestation of waterlogged areas

The soil has to be drained and cultivated over the whole area to be afforested. European aspen, hedge maple, alder, Norway maple and other trees will provide biological drainage and biological cultivation of the soil. The target species are identified according to the conditions of each site: Norway spruce is a good species to grow at waterlogged sites that have been drained. Plants (4 to 5 years old) are usually set by the hillock and mound planting methods.

Afforestation of salinised soils

At extremely salinised sites soil-improvement measures have to be taken to reduce the salt content of the soil. The pioneer species showing the best tolerance to salinity include the tamarisk, oleaster, hazel, black thorn and others; locust, black walnut, hedge maple and some cultivated poplars are also tolerant to salt. The key target trees for such sites are oaks, some poplars, olive ash *(Fraxinus excelsior)*, Scotch elm and Russian white elm *(Ulmus laevis.)*

Older plants (oak: 3−4 years old) should preferably be used for afforestation at salinised sites, though in the pioneer species one-year old plants may also be suitable. The target species should be planted at a spacing of, e.g., 1.25−1.0 m, poplars in rows 5−8 m (or even up to 10 m) apart, shrubs being planted between the rows.

9.5 Riparian stands

Riparian stands are special-purpose stands of shrub or tree species growing along waterways or lining the banks of water reservoirs.

The majority of riparian stands today have developed as a result of succession which can, for example, be very clearly observed on drainage channels with a permanent water flow: weed and grass associations first spread along the banks and are soon followed by self-seeded woody species, especially alders and willows. The seeds are brought down with the water and catch on along the banks within the reach of the moderate fluctuation of the constant flows. Riparian stands first develop within a comparatively narrow strip inside the channel just above the water level. Later on, further individuals catch on farther from the water level.

If a natural channel is too shallow, the water may overflow the banks to flood the adjacent land from time to time, so the riparian zone may become heavily waterlogged, providing good conditions for the development of alders and willows which first overgrow the banks but later spread farther, deteriorating the adjacent land.

Natural development of riparian stands is usually undersirable because they obstruct the flow, affect the microclimatic conditions along the water course and worsen the conditions for the cultivation of the land. The trees and shrubs that grow at the level of permanent flow grow into the channel, causing cross circulation of the water which damages the banks. Such unshaped trees, often growing in dense groups, catch objects floating on the water, narrow the flow profile and impound the water, causing it to spill over the banks. Even from the aesthetic point of view, dense wild riparian stands are of no benefit to the cultivated landscape.

If riparian stands are to fulfil their mission adequately, certain conditions have to be observed during their formation and maintenance.

9.5.1 Riparian stands along water courses

9.5.1.1 Mission of riparian stands along water courses

Tree and shrub stands are formed and cultivated to protect water courses which are important from the farming point of view. Their key mission is to protect the

slopes of the channel against damage that may be caused by flowing water. If properly managed, they also control the overgrowing and silting up of the channel and exert a favourable influence on the general environmental conditions in the farmed landscape.

Control of the damaging effects of the water flow

Trees and shrubs growing along the water courses protect the channel slopes. Their root system grows through the soil profile and the densely branched and interwoven roots and rootlets fix the soil particles (Fig. 223). The aerial parts of the trees and shrubs increase the roughness of the banks, thus reducing the velocity of the flow to nonscouring levels and keeping the stream-line away from the endangered banks.

In water courses in hilly and submontane areas, they also keep the moving gravel away from the slopes of the channel.

Controlling the overgrowing and silting of the channel

In flat areas in farmed regions, riparian stands also have the important mission of suppressing the growth of herb associations on the banks and aquatic flora on the bottom of water courses. Herbs can thrive if they are allowed to grow under warm conditions and with exposure to light. Shading reduces such conditions: the light-demanding vegetation will disappear from the banks and the only species that will remain are those tolerating shade and low temperatures. A similar effect

Fig.223 The root system of the alder (Photo L. Novák).

is exerted on the aquatic vegetation. During the growing season the flow rates and water levels in the channel are low, allowing the water to be warmed to a temperature favourable to the growth of many aquatic plants: sweet sedge, reed mace, watergrass, water plantain and a number of others. These aquatic plants then produce dense stands along the banks and on the bottom below the water table (Fig. 224). The wetted profile is heavily roughened and the plants intercept the small soil particles suspended in the water which further reduce the flow profile. The roots of the aquatic plants grow through the mud deposits and prevent the water from carrying them away at higher flow rates.

The overgrowing and silting flow profiles of water courses flowing through farmed areas and of drainage channels need frequent maintenance interventions. The need for maintenance can be reduced by forming riparian stands of species that will shade the channel, cool the water and control the growth of aquatic plants. This favourable effect of riparian stands was demonstrated by NOVÁK (1977) and ŠKOPEK (1977) who studied the effect of riparian stands along canals and water courses in typical agricultural watersheds. They found that shaded water courses were more or less free from overgrowing aquatic vegetation (Fig. 224) whereas the bottoms not protected by riparian stands were entirely overgrown and silted by sediment (Fig. 225). ŠKOPEK (1977) demonstrated that shading reduces the density of grass and herb vegetation on the bank slopes: in the upper part of the slopes the density of such stands is reduced to 55% and at the toe of the slope to 35%, compared with 100% on the banks of nonshaded water courses.

Effect of riparian stands on the self-cleaning capacity of water courses

The organic substances contained in polluted water are broken down by polysaprobic bacteria which attach themselves to the submerged objects. Riparian stands create favourable conditions for the self-cleaning process by allowing these bacteria to attach themselves to the roots of the trees, submerged branches, etc. However, if the channels are overshadowed by a very dense riparian stand, this process may slow down and the development of the bacteria may be reduced.

The environmental function of riparian stands

In a farmed landscape with only sporadic occurrences of trees, riparian stands contribute to maintaining the necessary environmental balance: they provide shelter to animals, are a source of food for some organisms and provide conditions for their reproduction. BAUER et al. (1980) believe that riparian stands are important for insects, e.g., for plant pollinators (humble bees, solitary bees) and for the predatory insects used for biological pest control, as well as for the development of some vertebrates, including game animals. All these organisms need stands containing a significant proportion of well-developed older trees.

9.5.1.2 Influence of riparian stands on the flow rate in a channel

ZELENÝ and JAŘABÁČ (1975) compared the flow rates in channels having trees on their slopes and in others having no riparian stands. As with the stands of bushy willows, tree stands were found to control water flows above the banks and above the bottom and also the cross-sectional velocity and the flow area (Tab. 84). The extent of this effect depends on the density of the stand. The data in Tab. 84 show

Fig.224 The channel of the Cerhovický brook shaded by riparian stand (Photo V. Škopek).

Fig.225 Channel of the Cerhovický brook overgrown with vegetation (Photo V. Škopek).

that the optimum density of the trees (to influence the flow conditions in the channel) is 0.22 trees per m^2.

Knowledge of the degree of roughness is needed to evaluate the flow rates in the channels. However, the optimum degree of roughness is difficult to determine because it depends on the structure and density of the riparian vegetation and on the ratio of the height of the vegetation inside the water profile to the depth of the water. ZELENÝ and JAŘABÁČ (1975) derived the data on the coefficient of roughness in natural riparian stands of trees, as shown in Tab. 85.

Table 84

Effect of tree stands on flow conditions (ZELENÝ-JAŘABÁČ, 1975)

Type of stand	Flow velocity inside stand reduced by %	Flow velocity above bottom increased by %	Mean cross-sectional velocity increased by %	Flow area increased by %
Very dense stand (0.67 trees per 1 m^2)	28	16	14	19
Thinned stand (0.4 trees per 1 m^2) (unattended natural stand)	39 (low flow rate)	1	10	16
Stand of optimum density (0.22 trees per 1 m^2) (well maintained natural stand)	24	9	5	5

On the basis of literature data, ŠKOPEK (ŠKOPEK and NOVÁK, 1977) recommends the following roughness coefficient:

one- to two-row riparian stand without undergrowth:

$n_{min} = 0.035$, $n_{mean} = 0.050$, $n_{max} = 0.075$

nonuniform riparian stand along the whole length of slope:

$n_{min} = 0.080$, $n_{mean} = 0.100$, $n_{max} = 0.120$.

PETRYK and BOSMAJIAN (1975) subjected the roughness of a channel overgrown with taller vegetation to theoretical analysis on the basis of an analysis of the tractive forces acting on the surface of the wetted profile and on the plants inside the profile in the channel. As shown in Fig. 226, the following tractive force develops inside the channel between profiles 1 and 2:

$$T = T_d + T_v \quad \text{(N)}, \tag{9.1}$$

where T_d – the part of the tractive force transmitted by the channel walls (N),
T_v – the part of the tractive force transmitted by vegetation (N).

Equation (9.1) can be written as follows:

$$\varrho Slig = \tau_o Ol + \sum T_{v_i}, \tag{9.2}$$

where ϱ – water density ($kg \cdot m^{-3}$),
S – flow area of the profile (m^2),
l – length of the channel (m),
i – slope of the water level,
g – acceleration of gravity ($m \cdot s^{-2}$),
τ_o – shear stress on the walls of the channel (Pa),
O – wetted perimeter (m),
T_{v_i} – tractive force acting on the ith plant (N).

In the equation for the shear stress on the walls of the channel,

Table 85

Degrees of roughness of riparian bush and tree stands (ZELENÝ-JAŘABÁČ, 1975)

Type of stand	hydraulic radius R (m)	n
One-year old dense willow stand, grown from willow covering, spacing 0.1 × 0.1 m	0.420 0.450 0.460	0.039 0.038 0.040
One-year old thinned willow stand grown from willow cover, spacing 0.2 × 0.2 m	0.410 0.450 0.480	0.039 0.037 0.036
Three-year old natural stand grown from willow covering, irregular growth and spacing	0.423 0.494 0.507 0.551	0.029 0.025 0.022 0.024
Very dense tree stand of willow and alder, trunk diameter 0.2 to 0.3 m, regular spacing (0.67 trees per 1 m^2)	0.281 0.480 0.580	0.040 0.040 0.042
Thinner tree stand of willow and alder, trunk diameter 0.2 to 0.3 m, regular spacing (0.4 trees per 1 m^2)	0.337 0.448 0.479 0.533 0.580 0.610	0.038 0.025 0.044 0.039 0.049 0.059
Natural 45-year old tree stand of alders (*Alnus glutinosa* and *A. incana*) and willow, trunk diameter 0.2 to 0.3 m, irregular spacing (0.22 trees per 1 m^2)	0.349 0.399 0.451 0.418 0.456	0.040 0.037 0.032 0.049 0.031

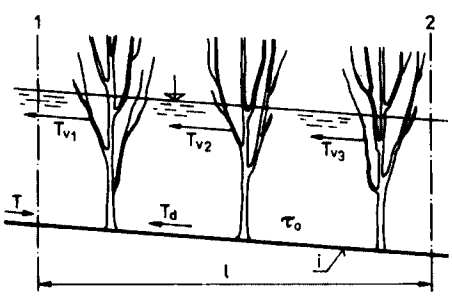


Fig.226 Effect of vegetation on roughness (after PETRYK and BOSMAJIAN, 1975).

$$\tau_o = \varrho R i g \qquad \text{(Pa)}, \tag{9.3}$$

i can be substituted by an expression derived from the Manning equation

$$v = \frac{1}{n_d} R^{2/3} i^{1/2} \qquad (\text{m} \cdot \text{s}^{-1}), \tag{9.4}$$

where n_d – roughness coefficient of the channel walls,
R – hydraulic radius (m).

Thus the equation for shear stress will have the form

$$\tau_o = \varrho \frac{v^2 n^2}{R^{1/3}} \qquad \text{(Pa)}. \tag{9.5}$$

The tractive force acting on each plant is expressed by the equation:

$$T_{v_i} = \frac{C_v \varrho v^2 S_i g}{2g} \qquad \text{(N)}, \tag{9.6}$$

where C_v – coefficient applying to vegetation ($C_v = 1.0$),
S_i – projection of the area of an ith plant into a plane perpendicular to the direction of flow (m^2).

Inserting equations (9.5) and (9.6) into equation (9.2) will produce the equation

$$\varrho S l i g = \varrho \frac{v^2 n_d^2}{R^{1/3}} g O l + \frac{C_v \varrho v^2 \sum S_i g}{2g}. \tag{9.7}$$

from which, after adjustment, the equation for average channel velocity can be derived:

$$v^2 = \frac{i}{\dfrac{n_d^2}{R^{4/3}} + \dfrac{C_v \sum S_i}{Sl \cdot 2g}} \qquad (\text{m} \cdot \text{s}^{-1}). \tag{9.8}$$

Comparison of equation (9.8) with the Manning equation for average channel velocity

$$v^2 = \frac{i}{\dfrac{n_d^2}{R^{4/3}} + \dfrac{C_v \sum S_i}{Sl \cdot 2g}} = \left(\frac{1}{n}\right)^2 R^{4/3} i, \tag{9.9}$$

in which n is the roughness coefficient comprising the effect of the roughness of the channel walls and of the vegetation, will allow one to derive the equation for the total roughness coefficient of the channel slopes:

$$n = \sqrt{n_d^2 + \frac{C_v \sum S_i}{S \cdot 2g} R^{4/3}}. \tag{9.10}$$

Fig 227 shows the nomogram for the determination of the roughness coefficient of a one-row riparian stand with trees or shrubs 2.5 m apart, the stand being located 0.5 m above the stable water level during the growing season.

9.5.1.3 Species composition of riparian stands

The species composition of riparian stands depends on the environmental conditions of different species and on the nature of the site. The most important species include the alders *Alnus glutinosa* and *A. incana*, European ash, maple, elm, poplar and willow; beach, hornbeam or linden can also be used. The best shrubs for riparian stands include the bird cherry, hazel, hedge maple, guelder rose, spindlewood, woodbine and buckthorn which form the understorey of high forests and whose roots penetrate through the surface layers of the soil in the banks, thus contributing to a good protection thereof against water erosion.

When designing the species composition of riparian stands, it is necessary to take into account each species' key properties, controlling their growth and development on the banks of water courses. These include, in particular, the shape of the root system, resistance to damage, capacity for regeneration after damage and resistance to floods. Considering the main mission of riparian stands – to protect the banks against water erosion – the best species to be used for this purpose will be the deeply rooting trees and shrubs whose roots are either amply ramified or heart-shaped.

The trees growing on the banks of water courses are exposed to the pressure of the flowing water which exerts stress on the roots; however, this stress can only cause damage to young stands (10–15 years) whose root system is not sufficiently branched to provide adequate protection to the banks. In addition to the pressure of the water, the trees and shrubs on the banks have to resist the pressure of the material transported by water and the pressure of floating ice. The danger of damage to the trees and shrubs is great on gravelly water courses where the rough gravel abrades the trunks of mature trees, damages the bark of young trees and can break and remove the newly set plants. Further damage is caused in the places where gravel is deposited: the gravel may sometimes be deposited in thick layers, covering the trunks of the trees to a considerable height.

Even greater damage can be caused by moving or building-up ice floes. The floes abrade the bark of the trees and can damage not only the thin trunks of young trees (which will be forced into the bank under the weight of the moving ice and are usually "peeled" along their whole length) but also the old trees whose bark is much stronger. Though in the latter case the bark is only abraded at the level of the ice flow, it is usually removed together with the cambium, thus causing really great damage.

The main trait underlying the resistance of the woody species to such damage is their regenerative capacity – their ability to renew or replace the parts damaged by abrasion, breaking, drying or embedding in sediment. The majority of deciduous species are able to send forth new sprouts from their adventitious buds and the sprouts will take root. This is typical of willow, alder, poplar and ash. The stands growing in riparian stands are exposed to flooding for certain periods during which they will have a poor supply of oxygen from the soil. The species tolerant to flooding include willow, alder, black poplar, aspen, English oak, elm,

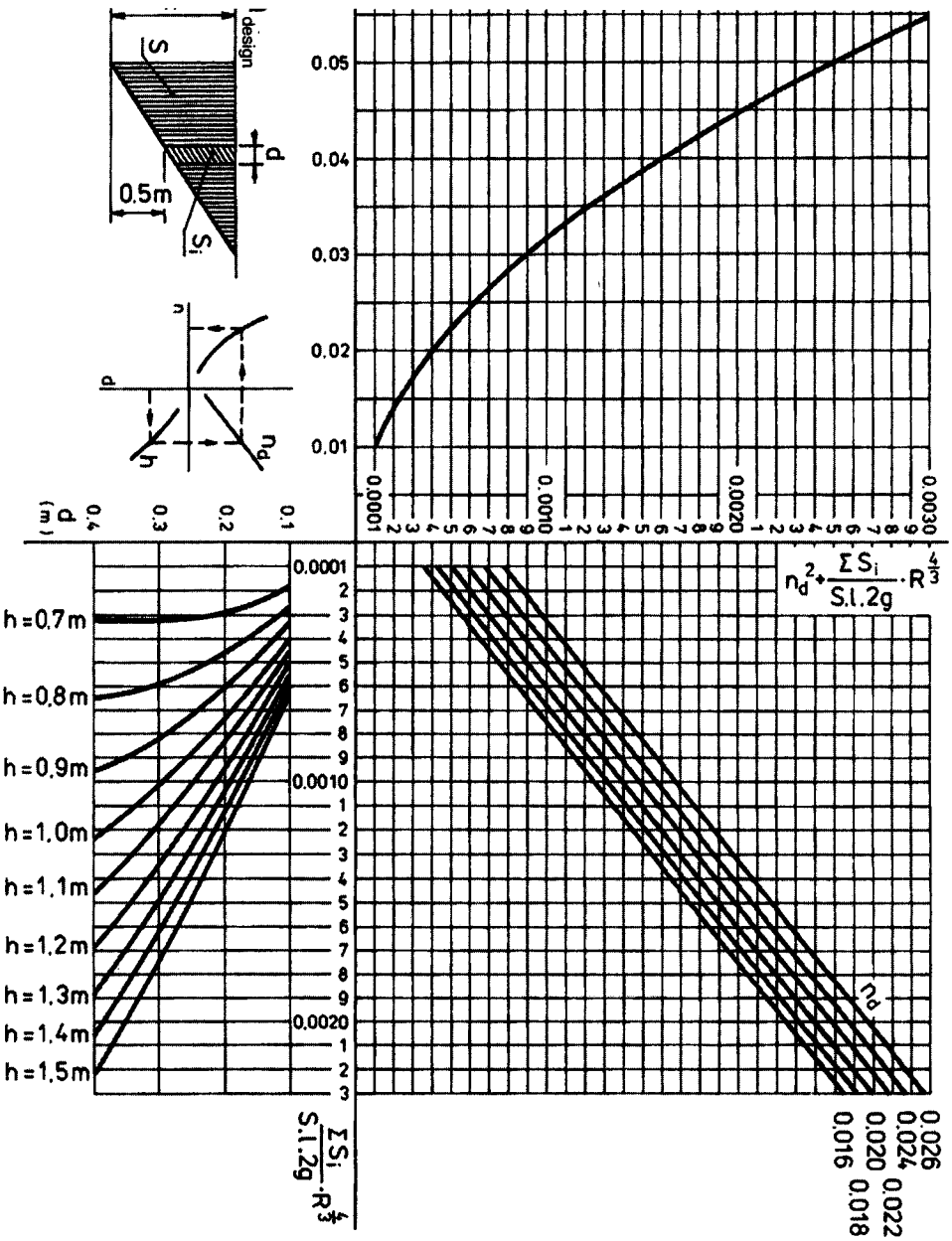


Fig.227 Nomogram for the determination of the roughness of a single-row riparian stand.

linden and Norway maple. When they have shed their leaves, deciduous trees can withstand flooding more easily. The different species can survive flooding for the following periods (rough estimates, only for general orientation):

– *during growing season*: willow 20–30 days, alder 15–20 days, oak, ash and poplar 10–15 days;

– *outside growing season*: willow 30–60 days, alder 20–30 days, oak, ash and poplar 15–20 days.

The woody species that can be used in riparian stands have the following characteristics.

Alder (*Alnus glutinosa* and *A. incana*). During the early stage of development, alders form a shallow and poorly branched root system. During the period around the 20th year of age, the development of the side roots slows down and the vertical roots become stronger. The alder is able to protect a bank within a length of 2–3 m. The thick anchoring roots partly grow through the banks even under the water level (0.3–0.4 m at the maximum). *Alnus glutinosa* normally occurs on loamy and gravelly soils in lowlands and hilly areas up to an altitude of 700 m above sea level. *Alnus incana* grows on gravelly soils in submontane and montane areas.

European ash. This tree forms thick roots growing aslant down the slope and when they reach the water course they spread further along the water level. These roots send forth a dense network of thinner vertical roots interwoven with still thinner horizontal and slanting roots. The root system reaches down to a depth of 1.4 m and the horizontal arms sometimes reach as far from the thrunk as 15 m. The area of occurrence of European ash extends from lowlands up to mountain areas 900–1100 m above sea level.

Sycamore maple. Its dense root system reliably binds the soil within a range of 2–3 m. The thick anchoring roots reach to a depth of 1.5 m and the roots growing laterally may grow to a distance of 10 m, though at such a distance they do not have a sufficient number of thin roots to be able to protect a bank adequately. Sycamore maples occur in hilly regions up to an altitude of 1500 m above sea level. Lowlands and lower-altitude hilly regions are populated by Norway maple.

Linden. Linden trees produce a thick root system which thoroughly binds the bank up to a distance of 2 m from the trunk. The thick anchoring roots may reach under the water level (0.3–0.4 m at the maximum). The large-leaved linden occurs at altitudes of 500–700 m above sea level and the small-leaved linden grows in lowlands and hilly regions up to 600 m above sea level.

English oak. Its root network grows densely throughout the bank within a distance of 2–2.5 m. The long and thick anchoring roots may reach farther from the trunk but are not sufficiently ramified to protect the bank reliably at such a distance. The English oak grows in lowlands and hilly regions up to an altitude of 400–500 m above sea level.

White willow. It forms a shallow root system consisting of very thick roots. It grows in lowlands.

Poplar. Poplars have a large root system, amply ramified, reaching to a great depth and width. They are resistant to abrasion damage.

Elm. In the early stage of its development the elm develops a taproot which later changes into a heart-shaped root system with thick side roots. The elm

Ulmus carpinifolia grows in lowlands and hilly regions up to an altitude of 400 – 500 m above sea level, *Ulmus laevis* grows in lowlands and *Ulmus scabra* occurs in hilly regions up to altitudes exceeding 1000 m above sea level.

The proportions of the areas covered by the different species depend on site conditions and on the natural association to which each particular species normally belongs. The objective is to produce proportions of species that would correspond to naturally developed stands. Information from forest typology can be used for designing the composition of target species of planted riparian stands. Considering the sets of forest types, the following associations would fit into the site conditions of riparian stands:

Softwood (poplar – willow) riverine forest: occurs in flat lowlands which are flooded once or more times a year and in which it grows on sandy, gravelly, loamy soils and loamy sands. The target species structure, expressed as percentual proportion of each species in the given area, is as follows: poplar 40, alder 40, willow 20, ash sporadic. The poplars include *Populus nigra, P. alba, P. canescens* and the willows in such stands include *Salix alba. S. fragilis* and their hybrids;

Hardwood riverine forest: grows on chernozem or brown-earth types of loamy sands to clay loams along rivers at altitudes of 140 to 260 m above sea level. The composition of target species includes English oak 50, ash 20, linden 10, maple and poplar sporadic. When such a forest is being planted it is possible to add hornbeam, bird cherry and hedge maple; red dogwood, spindlewood, hazel and alder buckthorn can be planted as undergrowth;

Brook ash forest: occurs on the brown-earth types of sandy loams to loamy sands on a gravel substratum. The target species structure includes ash 40, oak 20, alder 20, elm 10, maple 10, linden sporadic;

Brook ash – alder forest: occurs in the inundated areas along rivers in hilly areas where it grows on alluvial brown soils or on brown and humic gley. Target species structure: alder 60, ash 40, maple sporadic;

Grey alder forests: widespread in mountain areas on nondeveloped alluvial loamy sands to sandy loams, gravelly and stony soils. Target structure: grey alder 80, sycamore maple 10, ash 10, elm and black alder sporadic. The undergrowth consists of alder buckthorn, bird cherry, auricled willow, sallow and mountain ash.

9.5.1.4 Spatial structure of riparian stands

The design of the spatial structure determines the distribution of the riparian stand on the banks of a water course.

Riparian stands will only constitute a perfect lining of channel banks if favourable conditions are provided for their undisturbed development.

The most important point is that the bottom must be stable. If the bottom deepens, the growth of the trees' or shrubs' root system, controlled by the local conditions, is not able to adapt to the changes, so the toe of the bank loses support and the water severely damages the bank under the level of the roots.

Like any other method of bank lining, the root system of woody species has only a limited effect. Hence, in water-course regulation schemes, it is necessary to adjust the direction of flow so as to reduce the strength of the impact the water may have on the banks. Any unnaturally sharp bend of the channel exposes the roots of the trees which then fail to protect the bank adequately; in addition, the change in the

flow profile produces eddies which further disturb the flow and damage the channel.

Each tree or shrub must be well located within the flow profile so as to be able to fortify the bank adequately without disturbing the flow of water. Those growing close to the toe of the slope of the bank do not have sufficient space for their roots which then largely grow into the banks perpendicularly to the axis of the flow, leaving only some of the roots to grow along the toe of the slope – hence, the toe of the bank is only protected to an insufficient depth, no protection being provided to the strip of the bank just above the slope toe: this is often conducive to the formation of bank scours. Such trees or shrubs grow too far into the flow profile, thus affecting the right direction of the flow (Fig. 228). On the other hand, the roots of trees growing too high above (or far from) the water are not able to reach effectively the zone requiring particularly intensive protection: hence, the most exposed parts of the stand are left unprotected.

Tree stands established to protect the slopes of a channel must have sufficient density because the trees' physiological properties limit their bank-protection performance – the nature of each species' root system allows the roots to reach effectively only to a certain limited distance. To protect the bank reliably, the distance between the trees or shrubs must be adapted to the shape and extent of the root system. Thin riparian stands may leave some places unprotected and exposed to the scouring action of the water at increased flow rates. Bank scours develop in such places, reaching up to the nearest well-rooted and anchored tree or shrub (Fig. 229). If the tree at the end of the scour is rooted inside the toe of the bank, it cannot withstand the impact of water which cuts its way around it, extending the bank scours farther to a place with a compact riparian stand.

Depending on the level of permanent water flow during the growing season, riparian stands, whose basic mission is to provide stability of the banks (Fig. 230), are established either as single-row or multi-row stands and in some cases they may even cover a compact area. To provide effective protection of the bank toe it is recommended to plant trees or shrubs 0.50 – 1.10 m above the permanent flow level (Fig. 231). In streams where this level is about 0.1 – 0.3 m above the bottom, the woody species planted as mentioned provide sufficient protection even for the most heavily exposed bank toe which only needs a light temporary lining (e.g., a

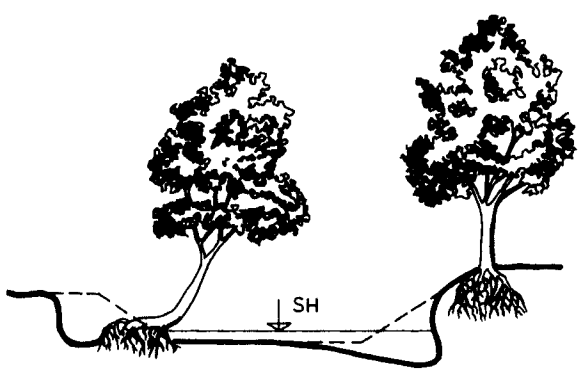


Fig.228 Inappropriate location of woody species (growing in the channel of the water course).

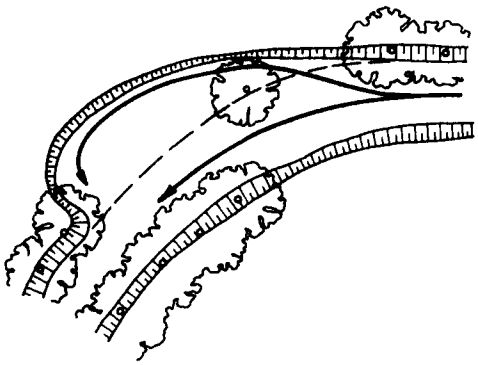


Fig.229 Formation of a bank scour in a riparian stand with gaps.

Fig.230 Protective riparian stand (Photo L. Novák).

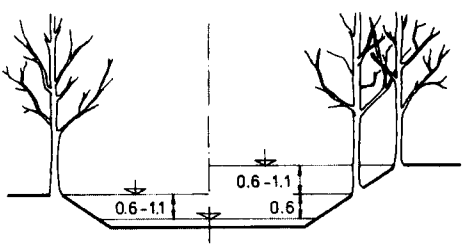


Fig.231 Optimum distribution of woody species on the slopes of the stream channel.

lath fence or low stone packing) to protect it before the new plants' root systems begin to protect the bank themselves. This approach is usually applied to small brooks and torrents. Where a shallow channel is designed (depth of up to 1.1 m), the trees may be situated outside the flow profile and be planted right on the edge of the slope. On larger water courses where the water level is too high above the bottom during the growing season, it may be necessary to use a stronger type of lining (e.g., stone packing) to protect the bank toe outside the reach of the roots. Trees can, again, be planted 0.6 – 1.1 m above this lining.

The stability of the banks on which trees (tall woody species) are planted also depends on the density of the stand. According to investigations conducted in natural water courses, the protective action of the trees' root systems does not reach very far from the trunk. Though the individual roots may grow far along the bank, they are not very amply branched so that their protective performance is not very effective. The closer to the trunk, the denser the roots; hence, the spacing of the individual trees in a riparian stand must be suited to their bank-protection performance. Concave banks are exposed to a greater pressure of the water, so the trees should not be planted farther than 1.3 – 1.7 m apart; on gentler bends the spacing may be up to 2.0 m. The same stand density can be used along straight segments, and on the convex parts of the bends it may be as much as 2 – 3 m. This principle applies not only to the first line of the trees (closest to the toe of the slope) but to all trees in the multi-row or compact area stands, covering the whole slope.

In areas with systematic drainage, the roots may grow into the drains where they will obstruct the flow of drain water. In such cases the stands of shrubs must be interrupted by a 5 m "empty" strip and tree stands by a 10 m strip, and the empty part of the bank has to be protected by a different method of lining.

For riparian stands designed to shade the channel and prevent weeds from overgrowing it, ŠKOPEK (1977) recommends that the following principles be respected:

a) Form a protective stand on both sides of small water courses and large drain and irrigation canals with a greater width between the bank edges: such stands will protect the channel even in cases of unfavourable orientation (SE – NW and SW – NE). To avoid undesirable shading of the adjacent farm land, plant the stands in the upper part of the channel banks beneath the edge of the bank.

b) Form a riparian stand along the southern or southeastern or southwestern bank of the channel of narrow water courses or small drain and irrigation channels.

The spatial arrangement of the riparian stand must allow for the maintenance of the water course. This is most important in stands heavily silted with fine sediment and those overgrown with aquatic, marsh and littoral vegetation. This mainly applies to special waterways in lowlands such as, for example, drains and canals. In such cases, the stand only consists of one line on one bank: the other bank remains free to enable mechanised maintenance. In the larger water courses where the machines have to be able to enter the channel itself, gaps have to be left in the riparian stand to allow the machines to get through. Single-row riparian stands with a compact undergrowth of shrubs are grown primarily on the banks of small water courses from lowlands to mountain areas. As to larger water courses,

it is as a rule possible to grow multi-row riparian stands which usually pass into compact stand covering areas unsuitable for farming farther from the banks.

The main mission of accompanying tree and shrub stands growing outside the flow area of the channel is to enhance the water courses' appearance, to include them organically in the landscape and support the water courses' general environmental functions. If the stands are to have a high functional and aesthetic value, their future mission must be known before they are planted. From the landscaping point of view, the most important factors are the selection of the species and the aesthetics of their distribution on the banks. The selection of the species for new stands must respect the general nature of the surrounding landscape and the distribution of other plant formations. Autochthonous species should always be preferred: introduced species often fail to combine organically with the species composition of the verdure dispersed in the landscape and with the forest stands.

Species have to be selected according to their appearance, including the formation of the crown, the size of the trunk, colour of the leaves and mode of insertion of the branches. A good combination of species will allow of making use of contrasting features of aesthetically valuable species. The species composition of the different groups of trees and shrubs should preferably combine large and wide-crown trees with the slim forms of other species (e.g., oak, maple and linden with alder or ash with birch). It is really necessary to avoid the dullness of stands of equal species (e.g., alder) and equal age with no height differences. Where a water course touches a forest stand, the strip of the riparian vegetation should not "dissolve" in the forest: for example, in a coniferous forest the trees along the water course should have light-coloured foliage, e.g., willows or ash) which would contrast with the dark conifers.

Riparian stands "highlight" the line of the water courses and divide the landscape into smaller closed units which enhance its aesthetics. This is particularly important in landscapes where there is a lack of other dispersed tree-type verdure. Of course, in fact, any association of tall plants will be of aesthetic value in an otherwise monotonous landscape, but the aesthetic impression can be enhanced by an adequate combination of species and also by a good spatial arrangement of the trees or shrubs along the water course. The best approach is to irregularly alternate groups of trees of different heights and combine them with strips of shrubs or shorter trees. Such stands do not form nontransparent masses: on the contrary, they open vistas of outstanding features of the landscape and in flat areas they offer views of other, more distant, landscapes (Fig. 232).

Shrubs have a special position among the stands lining water courses. Shrubs break the height uniformity of trees and provide continuity with other groups of verdure dispersed in the landscape. In farmed areas, the riparian stands should not be unnecessarily wide (so as not to affect the growth of cultivated crops): the distance from the adjacent fields should not be less than 1 m for shrubs and 3 m for trees.

9.5.1.5 Formation of riparian stands

Riparian stands can only be established on the banks of those water courses along which the conditions are favourable for the growth of woody species. On regulated water courses such conditions are provided by the formation of a stable

Fig.232 Riparian stand well distributed into layers (Photo L. Novák).

channel-bed gradient with no danger of bottom deepening and by realignment with the elimination of all sharp bends. Not all nonregulated water courses can offer such conditions for the trees and shrubs; sometimes the existing channel must be adjusted before planting the trees or shrubs (removal of sediment from the channel, stabilisation of gravel terraces, securing of bank scours, removal of deep scours in the bottom, minor local realignments, local structural or vegetational reinforcement of the toe of the bank, etc.) (Figs. 233 and 234). All this having been done, a riparian stand able to protect the bank adequately can be planted. Such stands are established by planting stronger plants, cuttings, osiers and stakes. It is possible to plant 4- to 6-year old saplings and where the conditions are exceptionally favourable, 2- to 3-year old transplants can be used, either naked or with rootballs; containerised plants can be used in less favourable periods of the growing season.

Stakes, osiers and cuttings of poplars and willows are good material mainly for establishing the accompanying riparian stands on extensive gravel deposits. The planting material should have the following dimensions:

stakes: 1.5−2.5 m long
30−60 mm thick at the lower end

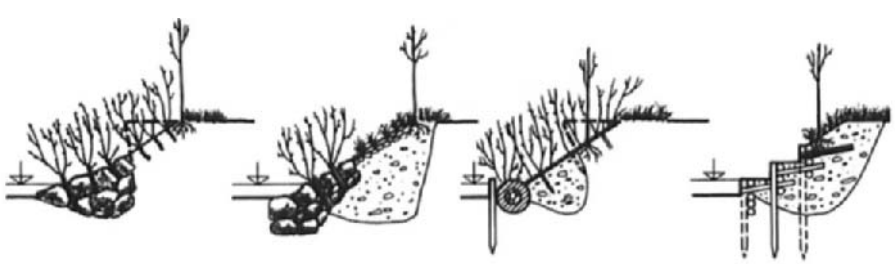


Fig.233 Biostructural control of bank scours.

osiers: 0.6 – 1.5 m long
10 – 20 mm thick at the lower end
cuttings: 0.2 – 0.5 m long
8 – 20 mm thick at the lower end.

The soil must be prepared before planting. Channel slopes should be weeded by means of selective herbicides. If there is sod on the slope, holes large enough to accommodate the root system of the plant (0.3 m in diameter at the minimum) have to be cut. Depending on the extent of weed infestation, the slopes must be subjected to biological or mechanical preparation. Biological preparation means cutting the weeds in areas about 1.2 m in diameter and spreading them on the ground in a layer about 0.3 m thick: the sod will quickly break down under this layer and planting will be easy in spring.

Mechanical preparation may include tillage of the whole area (an exceptional practice, resorted to in heavily sodded areas), ploughing of strips 0.7 – 1.5 m wide, cutting the sod to make "windows" into which the plants will be set, or – in waterlogged areas – formation of hillocks or mounds.

The best planting practice is to set the plants in holes. The holes may be dug manually but it is better (especially in planting accompanying stands) to use mechanical hole diggers (operated by two men) or tractor-mounted planting augers. In row-type stands each plant is usually tied to a stake driven into the ground next to it.

The cuttings, osiers and stakes (the osiers and stakes sharpened to a point) are slipped into holes prepared by means of large dibbers. Osiers are set to a depth of about 0.3 – 0.5 m, stakes about 0.6 – 1.0 m. On sterile soils or gravel banks not having sufficient nutrients for the plants, the addition of some humic soil is necessary for the plants to take root. The following amounts of added soil are recommended:

about 10 kg for saplings,
about 1 kg for cuttings,
about 3 kg for osiers and stakes.

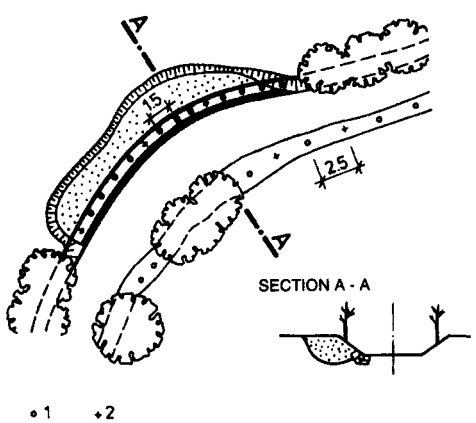


Fig.234 Stabilisation of a bank scour: 1 – alder, 2 – ash.

The best planting seasons are autumn and early spring. In autumn the planting may start as soon as the plants have shed their leaves and continue until the permanent frosts start. The spring planting season extends from the end of frosts to the onset of budding. Plants with rootballs can be planted from the end of August until the soil freezes and in spring until budding. Plants in containers can be planted almost all the year round except in summer and in the period during which the soil is frozen.

The density of newly formed riparian stands difers from that of mature stands capable of fulfilling their functions. The planting spacing is usually smaller but with each tending intervention (cleaning and thinning) the spacing is enlarged.

On small water courses in lowlands, it is best to plant a single-row riparian stand within the bank line or at a maximum distance of 1 m from the channel (Fig. 235). In the main part of such a stand, the tall deciduous trees are comparatively far apart and the shrub layer does not form a canopy. If a riparian stand is to be established along a channel consisting of a double-trapezoidal profile, it may be decided to form such a stand only on the outer slope of the channel (Fig. 236). If the stand is also designed to stabilise the slopes of a cunette, a single-line stand, consisting only of tall trees, may be formed in the bank edge thereof (Fig. 237).

On small water courses in hilly, submontane and montane areas, it is recommended (from the point of view of landscaping) to assume that channel slopes will be protected – with only minor exceptions – by a riparian stand of woody species. However, the protecting effect of riparian stand cannot reach as far down as the toe of the slope which, therefore, will need a different type of protection against the action of the water flow. Considering the development of the trees' or shrubs' root systems (which will, after some time, reach close to the level of low flow rates), the best types of lining are stone packings and ripraps (both vegetated and nonvegetated), lath fences and others. Plants of woody species have to be set at an adequate distance above such a lining (Fig. 238).

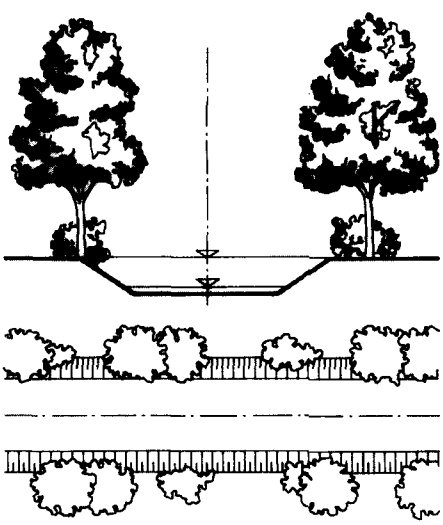


Fig.235 Single-row accompanying riparian stand along small water courses in lowlands.

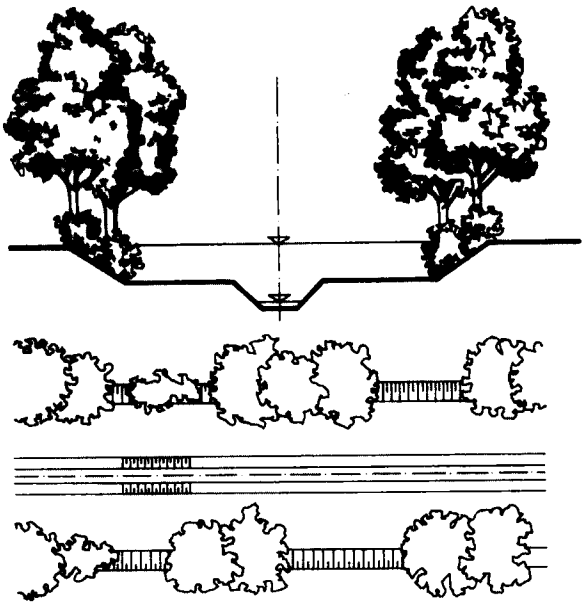


Fig.236 Accompanying stand on the outside slopes of a compound trapezoidal profile.

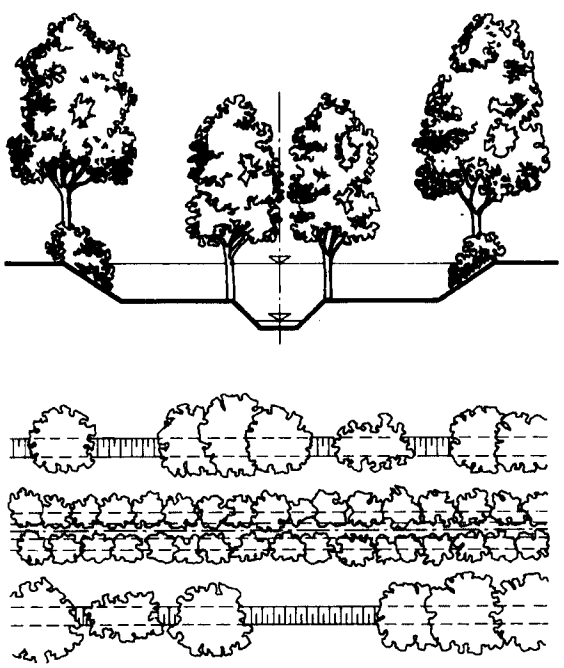


Fig.237 Accompanying stand complemented by a shelter stand in the banks of a cunette.

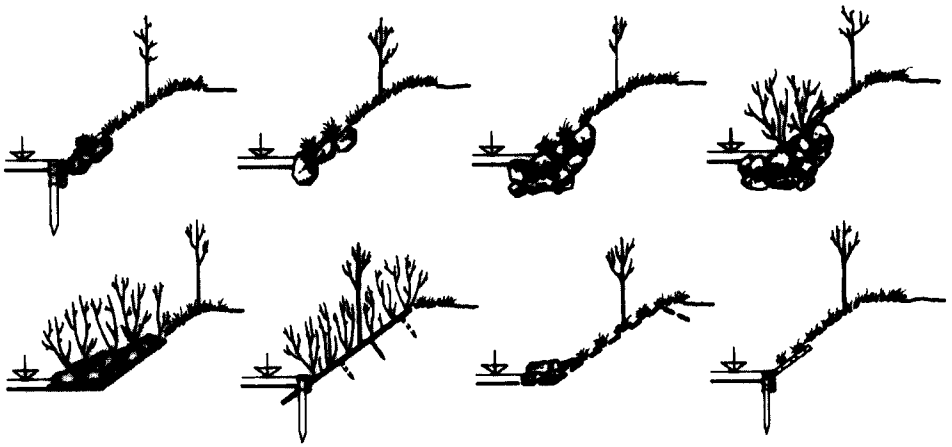


Fig.238 Establishment of a riparian stand above the biostructural lining of a channel slope.

The newly formed riparian stands need care. Watering (5 to 20 litres per plant) will protect them against drought. Depending on the weather conditions, watering should continue until the first frost. During the first year after planting, the soil around the plant should be loosened twice to four times to reduce evaporation from the soil. However, the soil around plants on the slopes of a channel is left without loosening.

The newly formed stands also have to be protected against tall weeds which may suppress their growth (especially when the plants are shorter than the weeds) and compete with them for nutrients and water. The protection is based on weeding and on mowing of weeds once or twice annually. It is recommended to spread the mown material around the plants. MACOUN (1988) recommends using mulching biotextile sheets 0.65 × 0.65 m in size, cut from one side to the middle and spread around the neck of the root of the plant and fixed to the ground with stones, wooden stakes or steel hooks (Fig. 239). This measure can remain effective for three years.

Loosening, weeding and weed mowing are most needed during the first year after planting. If necessary, these measures are continued during the subsequent years.

To protect the plants against browsing by animals, their trunks may be wrapped in paper, reeds or wire netting, or may be painted with repellent chemicals. Electric fences are installed to separate newly planted areas from pastures.

After every flood, the plants on channel slopes have to be cleaned (sediment deposits and trapped objects must be removed). In spite of all the care given to the young plants, a perfectly established riparian stand is never achieved after a single planting. The withered plants must be immediately replaced by larger ones.

9.5.1.6 Maintenance and regeneration of riparian stands

To be able to reliably fulfil their mission, i.e., to protect the banks, riparian stands should be well maintained. Uncontrolled stands would grow too wild and could damage the channel bed. Optimum density and spatial distribution should

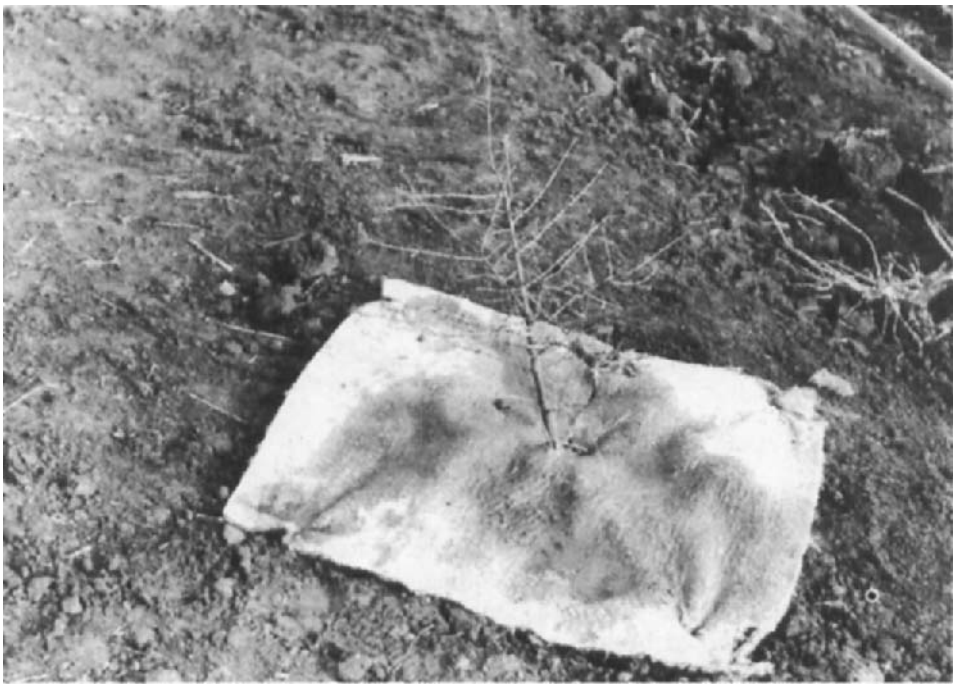


Fig.239 Protection of a plant with a mulching sheet (Photo Z. Macoun).

be maintained in an existing riparian stand to provide undisturbed flow of water along the channel. Trees growing too close to the level of standing flows, bent trees, trees reaching far over the water, those that are too old, dry or damaged and those affected by any other defect are removed. The trees growing at an optimum height above the water surface and providing optimum protection to the banks should be supported, though their growth may be suppressed. An example of riparian stand management is shown in Fig. 240.

Shrubs under a tall tree stand can only grow along the edge of the bank. Their presence in the flow profile is undesirable: they narrow it down and cause increased turbulence of the flow.

It is important from the viewpoint of bank protection that riparian stands should be compact. If there are gaps in the stand and some places in the bank are left unprotected, the banks are undermined, bank cavings develop and the flow line changes its regular course. It is therefore necessary to fill the gaps with new trees. Improvement of the existing riparian stands will require some local regulation of the water course itself. If the stream meanders excessively, it has to be slightly realigned and then the thicker trees growing too close to the water level must be removed, including even the stumps. Such places will have to be reinforced with stone packing or riprap.

The majority of the species growing in riparian stands are characterised by a very high stump-sprouting capacity. The presence of very dense saplings narrows down the flow profile. Their development has to be prevented by painting the stumps with suitable chemicals.

Fig.240 A tall riparian tree stand before and after the silvicultural treatment.

9.5.2 Riparian stands by reservoirs

Both natural and planted riparian stands along the shores of water reservoirs may have a number of functions, including (NOVÁK, et al., 1986):

- improving the stability of the banks by their root systems,
- protecting the banks against the action of waves by their aerial parts,
- reducing the effect of wind on the aeolian wave motion of the water level,
- helping to incorporate the water area of the reservoir into the landscape,
- providing verdure for the biologico-structural lining of the banks,
- reducing the erosive action of the surface runoff and transport of soil from the bank slopes to the reservoir,
- improving the reservoir's nourishing and reproductive potential if it is used for fish-culture purposes,
- utilising the wood mass produced,
- reducing the costs of bank protection and maintenance.

Depending on the key function of each stand, riparian stands can be subdivided into protective and accompanying stands.

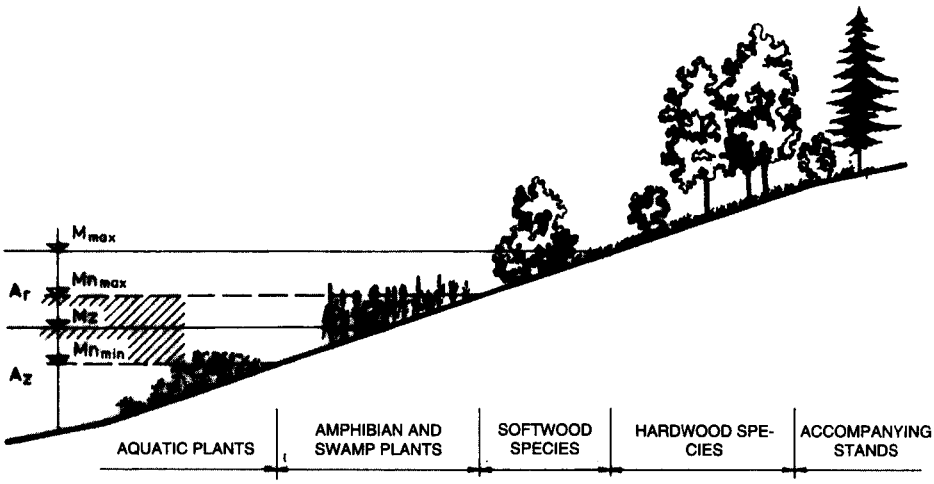


Fig.241 Vertical distribution of the different vegetation associations on the banks of water reservoirs (after NOVÁK et al., 1986).

Fig. 241 shows the vertical distribution of the different plant associations on the banks of water reservoirs:

– sublittoral zone: floating aquatic plants, submerged plants, both rooted and nonrooted (duckweed, Canadian waterweed, pond-weed),

– eulittoral zone: amphibian and swamp plants in the lower zone (water persicaria, watergrass, reedmace bullrush, sedge and reed) and softwood species in the upper zone (willows, alders and also herbs and grasses),

– supralittoral zone: hardwood species and grassland of meadow type in the lower zone, accompanying stands and protective forest belts in the upper zone.

9.5.2.1 Using riparian stands to protect the banks of water reservoirs

The banks of water reservoirs, exposed to abrupt fluctuations of water levels, rapidly yield to sliding along the newly wetted slide surfaces: in this way, the slope gradient of such banks adjusts to levels corresponding to the natural gradient of wetted earth. Extensive damage – formation of large cavings with a steep bank – may be caused by the waves due to the wind or larger boats. Such abrasion activity damages the (often very productive) fields adjacent to the reservoir. The loss of the soil is irreversible, which causes a permanent economic loss to the farm affected.

Opportunities to use vegetation for protection against the abrasion of reservoir banks are limited. The protective action of vegetation lies in the fixing of the soil by the root systems of the trees and shrubs (or herbs and grass) and in the buffering of the impact of the waves by their aerial parts. However, vegetation can act in this way only on banks of a low gradient where the abrasive activity of the water is not very intensive.

Protective growths can be formed in the aquatic-flora zone on the abrasion flat which develops within the stabilised slope under a large caving. The following

aquatic plants are suitable: reed, *Baldingera arundinacea*, reed-mace bullrush, sedge and true bullrush.

As to woody species, the bushy willows form the best component of protective vegetation. Willow growths are formed on slopes with gradients of up to 6−15°. When designed to serve as a protective belt, they are planted before the reservoir is filled and cover the strip of bank between the dead storage level M_Z and the maximum level M_{max}. However, this involves a danger: if the water level stays above the dead storage level for longer than 90−105 days, the willows in the flooded lower parts of the strip may die. The following bushy willows are the best choice for planting along the shorelines of reservoirs: *Salix cinerea, Salix fluviatilis, Salix triandra, Salix viminalis, Salix purpurea, Salix hookeriana, Salix rubra, Salix lucida* and *Salix interior.*

Tree species do not provide good protection of the banks exposed to abrasion and are not able to stop the progression thereof: hence, they are only established as accompanying stands above the maximum water level.

A diagram of the distribution of vegetation on the banks of water reservoirs (before the onset of abrasion), as proposed by ŠKOPEK (NOVÁK et al., 1986), is shown in Fig. 242.

On banks already damaged by abrasion (large abrasion cavings), it is first necessary to adjust the steep wall of the caving to a more gentle slope and to form the willow growths as vegetated structures which will also provide strong protection against the action of waves. A diagram of the distribution of the vegetation in a place affected by a large abrasion caving, designed by ŠKOPEK (NOVÁK et al., 1986), is shown in Fig. 243.

Structural and biostructural approaches are used to arrest the progression of abrasion cavings. The best structural approach to the fixing of the toe of low-abrasion cavings and to the lining of the upper part of the abrasion flat is to use stone packing. Riprap is the best structural solution for the toe of high-abrasion cavings.

However, this method, though effective, distorts the reservoir's shoreline because the vegetation is not able to cover up the unshapely surfaces of the lining structures.

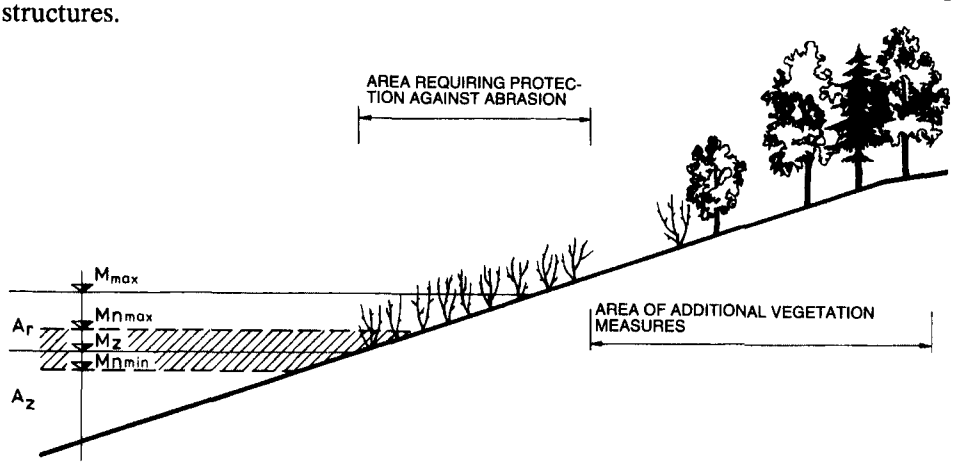


Fig.242 Diagram of the distribution of vegetation on the banks of reservoir before the rise of abrasion (After NOVÁK, IBLOVÁ, ŠKOPEK, 1986).

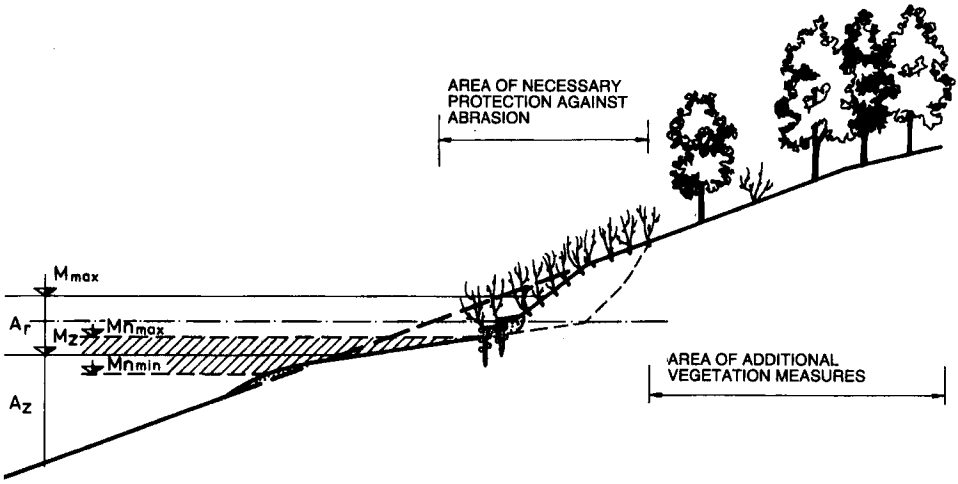


Fig.243 Diagram of the distribution of vegetation after the formation of abrasion scours.

The best approach is to use a biologico-structural solution. The vegetational features of such structures check the waves (which would otherwise run into the shore) and their aerial parts dissipate the action of the waves. Thus, the protective action of the vegetation effectively combines with that of the structure itself and, in addition, the vegetation offers the aesthetic advantage of covering the structure. Such vegetated structures may include vegetated loose rock covers (Fig. 244), two-row wicker fences (Fig. 245), vegetated log walls (Fig. 246), vegetated gabion mattresses, vegetated ripraps (Figs. 247 and 248) and vegetated stone packings (Figs. 249 and 250).

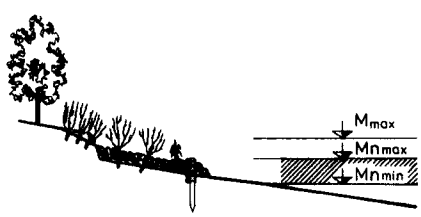


Fig.244 Stabilising an abrasion scour with vegetated loose rock cover.

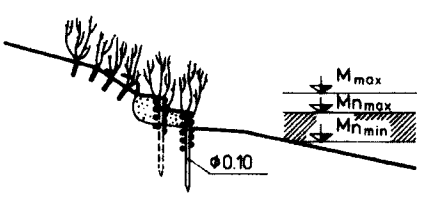


Fig.245 Stabilising an abrasion scour with a double wicker fence.

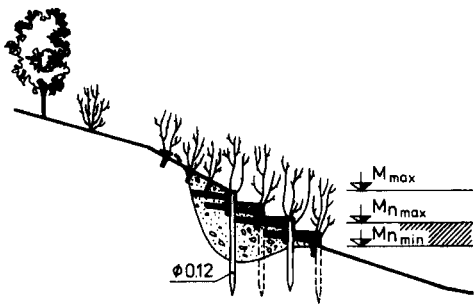


Fig.246 Stabilising an abrasion scour with vegetated lath fences.

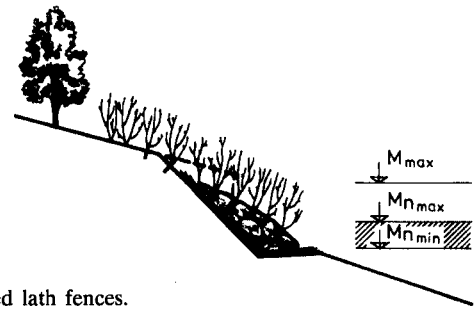


Fig.247 Stabilising an abrasion scour with vegetated riprap.

Fig.248 Vegetated riprap with mature willows (Photo V. Škopek).

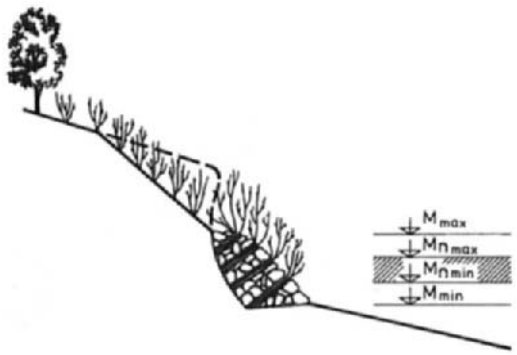


Fig.249 Stabilising an abrasion scour with vegetated stone packing.

Fig.250 Stone packing (Photo V. Škopek).

An example of using the biologico-structural approach to the protection of a reservoir bank heavily damaged by abrasion is shown in Figs. 251 and 252.

Fig.251 An abrasion scour on the Slapy reservoir, Bohemia (Photo V. Škopek).

Fig.252 Stabilised site – situation 10 years after the intervention (Photo V. Škopek).

9.6 Forest shelter belts

The favourable water management, soil protection and climatic action of forest stands is used for the protection of cultivated crops and agricultural land against the destructive action of wind. The narrow forest belts specially designed for these purposes include windbreaks and infiltration belts.

9.6.1 Windbreaks

Windbreaks are protective forest belts designed to protect farm crops against the destructive action of wind through reducing the wind speed.

The damage caused to crops can be divided into two categories. In arid areas, the warm drying winds deprive the crops of their moisture by intensifying the evaporation from the soil and through increased transpiration. The wind currents characterised by a lack of moisture, hence having a high saturation complement, are set in turbulent motion when they touch the rough surface of crop stands: during this process, the lower layer of the air, whose moisture content has increased through evaporation from the soil and through increased transpiration of the plants, mixes with the dry layers, thus continuously renewing the destructive action of the wind.

Further damage occurs in areas with cohesionless earths from which the fine soil particles are carried away by the wind (wind erosion, soil deflation): wind can cause this effect at wind speeds as low as 5.0 $m \cdot s^{-1}$ in the layer just above the soil surface.

Forest shelter belts have become widespread thanks to the research work conducted in Russia and the USA. The Russian scientist, DOKUCHAYEV, planted shelter belts during his investigations in the chernozem steppe areas already late in the last century. In the USA and Canada, shelter-belt planting projects started as a response to extensive losses caused by winds that had blown away fertile soils and by dust storms that had affected those countries in 1932–1934. The Russians started to plant forest-shelter belts on a large scale after 1938 and then again after the war, in 1948. Much work in this area was also done in the flat areas of Denmark, Germany, Belgium, Switzerland, France, Hungary and the Netherlands.

9.6.1.1 The favourable effect of windbreaks

Effect on windspeed

The basic protective action of windbreaks lies in their capability to buffer the wind speed which is reduced on both the windward and leeward sides of the windbreak belt (Fig. 253). An air cushion with a much reduced air movement forms on the windward side: the winds have to get across the air cushion thus losing much of their speed. On the leeward side, comparatively far from the windbreak (depending on its permeability and width) the wind may regain its original speed.

Depending on the strength of their action, windbreaks can be divided into two groups: permeable (blown-through) and impermeable belts.

Permeable (blown-through) belts. Permeable belts form an obstacle which is not entirely closed to the wind. The flow lines of the wind largely pass

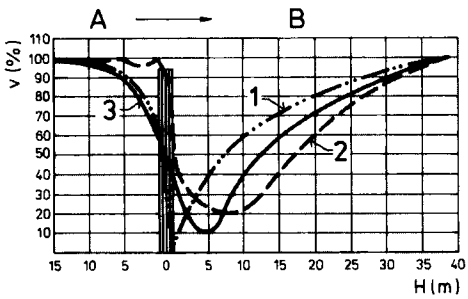


Fig.253 Effect of windbreak: reduction of windspeed. A – windward side, B – leeward side, 1 – impervious belt, 2 – permeable belt, 3 – semipermeable belt, H – distance from belt expressed as a multiple of belt height.

through the roughened space of the windbreak and lose speed through friction against the branches, stems and leaves. A minor part of the flow lines is lifted upwards by the windbreak, forms whirls and loses its original speed. On the leeward side these whirls descend and lose speed again as they impinge against the currents that have passed through the belt (Fig. 254).

Impermeable belts. Impermeable windbreaks form a closed obstacle, its whole profile being impermeable to the wind. This is due to the thick undergrowth under the tall trees whose crowns form a solid canopy. Only a minor proportion of the wind flow can make its way through the belt: the prevailing part of the air mass is lifted up to drop down again behind the belt onto the protected area where it meets almost no wind flows that have passed through the belt, so the wind is slowed only by the roughness of the belt's surface which thus controls the wind to a shorter distance than the blown-through belt.

According to the data recorded in Russia, it can be generally stated that:

– on the windward side of the windbreaks, the wind speed is reduced within a distance equal to five times the height of the belt. The greatest reduction is achieved with the impermeable windbreak where the wind speed is reduced to 60% of the original value on the windward side and to zero on the leeward side;

– on the leeward side, the action of the windbreak remains effective up to a distance equal to 20–40 times the height of the belt. Just behind the impermeable belt the wind speed is reduced to 10–20% more effectively than behind the permeable belt, but its action is effective over a shorter distance.

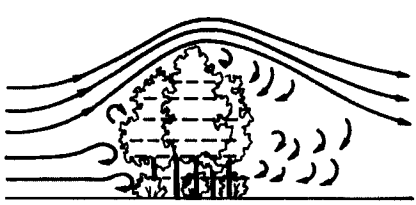


Fig.254 The air streamlines in a permeable windbreak.

Permeable belts do not act so intensively, but their effect is smoother, allowing the wind to regain its original speed only slowly on the leeward side, so their overall effect is greater.

Hence, the distance over which the windbreak is able to control the wind speed depends on its height and structure. Another important factor is its width. The proportions of all these factors must be carefully considered from the economic point of view (farmland necessary for the belt and the costs of planting and maintaining it).

According to research results recorded in Russia and Switzerland, a narrow forest shelter belt is more effective than a wide one. At a distance equal to 25 times the height of the belt, the wind speed behind a narrow belt is reduced to 80%, on an average, whereas behind a wide belt it is close to the original speed. This is due to the fact that on the leeward side of the belt the air flows are quickly forced down to the ground by the pressure of the upper air layers. In the case of narrow belts, which are more permeable than wide ones, only part of the wind is turned upwards, leaving much of the air mass to pass through the stand where it loses its speed.

Even a windbreak as narrow as one or two rows can provide some protective action. The effectiveness of a belt depends on its species composition, on the ramification of the trees and on the density of their canopy. Gaps in the stand have a considerable adverse action: they may increase the wind speed to as much as 120% of the original speed. Hence, any paths or roads inside the windbreaks must lead aslant through the belt.

Effect on air temperature

Forest shelter belts can both increase and decrease the temperature in the area they protect. During the forenoon, the air temperature between the belts is higher than in the exposed area, at noon the temperatures are more or less the same and from about three p.m. and during the night, the temperature in the sheltered zone is lower. During very hot days, the temperature in the sheltered zone next to the belt may increase by 6 – 7 °C which may be dangerous to some crops.

The average temperature is 1 – 2 °C higher in the protected zone than in the nonsheltered zone and in the impermeable stands the difference may be as high as 6 °C. The increase in temperature may have a favourable influence on the cultivation and yields of the crops but the reduction of temperature during the night may have an adverse effect.

Effect on atmospheric humidity

Humidity is indirectly proportional to the changes in air temperatures. During the first half of the day, the humidity declines but during the afternoon the trend reverses and the relative atmospheric humidity rises (the saturation complement falls). After sunset, as measurements show, the relative humidity is increased by 5% (but Russian data state that the increase may be up to 15%) and the absolute humidity can increase by 2 – 3 mm. The greatest air humidity is recorded in the area adjacent to the belt and near the soil surface. Humidity declines from the belt to the middle of the sheltered zone and from the ground upwards.

Effect on evaporation from the soil

Evaporation from the soil is a key indicator of the effectiveness of a windbreak. According to Russian data, evaporation on the windward side of the belts decreases to 35−45% of the evaporation levels in the open steppe; towards the middle of the sheltered zone it rises to reach 90% at a distance of about 50 m from the belt. On the leeward side, the effect of the belt on evaporation is stronger: at the border of the belt the evaporation is reduced by 2.5−3 times and then it successively declines with the distance from the belt. Some effect is still recorded as far from the belt as 300 m.

Effect on the deposition of snow

Windbreaks also contribute to the formation of water reserves in the soil, especially reserves of moisture from the winter precipitation. In the flat areas in lowlands with poor precipitation, it is necessary that the small amount of snow that falls in such places should be maintained on the soil and the wind should not be allowed to blow it away to places where such winter moisture would be of no use for the farmer.

Russian data indicate that windbreaks reduce the transport of snow by reducing the wind speed, thus contributing to a more uniform snow distribution over the area between the belts.

The interception and gathering of snow in and around the belts depends on the construction of the windbreak and on its width. Dense and impermeable belts cut wind speed abruptly so that much snow gathers at and inside the belt, whereas at a certain distance from the leeward end of the belt the snow falling over the belt may be blown away with the wind. Permeable belts do not form deep snow drifts and the weakened wind currents that can pass through such a belt are able to distribute the snow more or less uniformly over the area behind the belt. The thicker snow layer on the leeward side of the belt reaches farther into the fields, compared with the impermeable belts.

Excess width of the belt is also conducive to the gathering of much snow over the area of the belt itself and in its vicinity and the snow may often be drifted to considerable distances over the sheltered area. This irregular distribution of snow may be conducive to the frost killing of autumn-sown crops in places from which the wind had removed the snow; it may also be responsible for insufficient storage of winter moisture and, on the other hand, for the waterlogging of autumn-sown crops and delays in the spring cultural operations in the area adjacent to the belt where the snow drifts take a long time to thaw.

Windbreaks are also able to prolong the duration of the snow cover in the sheltered area by up to 30%, thus contributing to effective protection of autumn-sown crops against winter killing. From this point of view, the most effective windbreaks are the narrow ones, permeable in their lower part, which evenly distribute the intercepted snow over the area and contribute to improvement in moisture storage in the soil.

Effect on wind erosion

By reducing the wind speed, the windbreak also protects the fields against wind erosion which would otherwise cause considerable losses by removing soil particles

and exposing the roots of newly sown crops which may then be killed by drought or frost; further losses, on the other hand, are caused by the deposition of the blown soil particles which may cover large crop areas.

As the prevention of the removal of soil particles is the main mission of forest shelter belts, the belts must be formed so as to reduce the wind speed below the level at which the wind is able to move the particles. This is best done by narrow forest belts with an understorey of shrubs and with an about 40−50% permeability. The wind-breaking action of such belts is effective within a distance equal to 5 to 10 times the height of the stand on the windward side and 20 times its height on the leeward side.

Improvement of farm crop yields

If the windbreaks are to pay off, their overall effect should manifest itself in increased yields of the crops in the sheltered area.

As known from experience gained in the steppe regions of Russia, the yields may be 20−30% higher in the sheltered areas compared with open steppe in years with normal precipitation and 100−200% higher in dry years. This effect varies with crops: the highest yield increases are recorded in grasses (100−200%), followed by fruit orchards (50−75%) and cereals (25−30%).

Effect on insect and weed control

Forest shelter belts provide natural shelter for game animals and for the nesting of birds which play an important role in biological insect control. Different weed seeds blown by wind become attached on the ground parts of the windbreaks, on the soil surface, in shrub stands and on the branches of the trees. RIEDL (1966) recorded that 56,968 different seeds were caught during the month of January in a windbreak 100 m long and 1.3 m tall.

Landscaping role

Windbreaks are usually formed in landscapes poor in tree and tall shrub greenery. Hence, windbreaks also contribute to the aesthetic appearance of the rural landscapes, so it is important to use adequate tending practices to maintain the windbreaks as shapely as possible.

9.6.1.2 The adverse effects of windbreaks

The forest shelter belts may sometimes show adverse manifestations. The main drawback is that the windbreaks have to be planted on farmland. Another disadvantage is that the yields of crops grown close to the windbreaks are reduced because of shading and also because of competition for nutrients and moisture. Some species of parasitic fungi and harmful insects may also find favourable conditions for their development in the shelter belts.

9.6.1.3 Formation of windbreaks

Forest shelter belts should always be established where the soil surface is exposed to wind erosion and where the crops have poor yields owing to exposure to dry winds. Windbreaks are built in systems consisting of main belts across the direction of the harmful winds and additional belts perpendicular to the main belts. When

forming windbreaks, use should be made of the forests and other types of tree and shrub stands in the farmed landscape, including vegetation along water courses and roads. Depending or their levels of effectiveness, it is recommended to plant the main forest belts 300−400 m apart on sandy soils, 500 m apart on loamy soils and up to 600 m on heavy-textured soils.

Choosing the right species

The selection of the species depends on the site conditions and on the requirements to be met by the trees and shrubs in the windbreak. Since the areas in which the forest shelter belts are to be formed correspond to the forest types of various combinations with oak, the most frequent species will include *oak, hornbeam, linden, maple, alder, ash* and *elm.* All these species have the good characteristics needed to make a good windbreak. These include, first of all, perfect rooting and fixing in the ground to withstand a great wind pressure. It is also required that the trees' roots should not reach very far into the farm land. Species forming root tillers are entirely undesirable (*white poplar, aspen*).

Oak, maple and *linden* are the best trees to be used as the main species in windbreaks; alder and ash are preferred in wet flood-plain sites.

The main species should be combined with auxiliary species to form a stratified canopy structure: growing to a smaller height, the auxiliary species can effectively fill the space just under the crowns of the main species in the windbreak. *Linden, maple, elm* and *hornbeam* are the best trees to be used for this purpose.

Shrubs are an important component of the windbreak structure: they fill the space under the crowns of the trees and close the windbreak's border. They control the flow of air in the bottom part of the windbreak, intercept snow, control the movement of soil particles and weed seeds with the wind and provide a microclimate and protection for the main and auxiliary species before they form a canopy. The following shrubs are most frequently used in windbreaks: *woodbine, spindlewood, privet, dogwood, Caragana arborescens, guelder rose, meadowsweet, hazel* and *currant.*

Techniques of windbreak formation

The different species should be planted with due respect to the possibilities of mechanised treatment in rows, which are usually 1.5 to 1.7 m apart. Most of the species are planted, only oak can also be grown from seed on site. One- to three-year old transplants are most frequently used. Notch planting is the best planting method. Planting machines may be used in flat areas.

During the first 3−4 years after planting, the young trees require increased attention. Weeds which quickly spread between the rows of plants should be removed by shallow ploughing (using a plough or cultivator twice to four times annually). The plants need hoeing three times annually until they are able to form a canopy. Irrigation has to be provided to prevent any losses that could be caused by drought. Dead plants must be immediately replaced by advanced plants of the same species.

Special attention has to be paid to the protection of the plants against wild animals, otherwise they would cause disastrous losses. Chemicals are the main means of protecting the young plants against such losses.

Tending interventions have to be focused on maintaining the necessary degree of permeability of the windbreak. The best effect is attained with windbreaks having a permeable ground layer, consisting of shrubs which, however, grow and spread very quickly and have to be frequently thinned. In the auxiliary and main species all interventions will be limited to the regulation of growth and formation of the required canopy.

9.6.2 Infiltration belts

Infiltration belts are *forest belts* designed to protect farmland against water erosion. Their purpose is based on the ability of the soil under forest stands to intercept the surface runoff and transform it into subsurface runoff.

Infiltration belts also help to increase the reserves of underground waters, which is important during periods of drought for farmland downstream from the belt. It is also important that the belts protect the adjacent land against increased evaporation of water from the soil and that they catch the snow and support its uniform distribution. Like windbreaks, they also have a landscaping role: large stretches of arable land do not look very aesthetic in the landscape but strips of forest stands can divide them into more shapely units and enrich the landscape by dispersed tree verdure. The protective potential of such belts depends on the quality and thickness of the layer of surface humus, on maintaining the crumbly structure of the soil and on the species structure of the stand. Measurements of infiltration under forest stands showed that in impermeable soils the infiltration rate was 6.085 mm per minute and in permeable soils 9.1 m.

The species structure of the infiltration belts depends on the site conditions and on the requirements associated with the belts' mission. The selected species have to be able to root amply through the soil profile, thus also contributing to maintaining a crumbly structure with a high water-holding capacity. Deeply rooting species are the best, including for example oak, sycamore and Norway maple, linden, alder, ash, beech, pine and larch.

To improve the moisture retention capacity of the belts on steep slopes, obstacles to runoff are installed within them (live or dry lopping fences, lath fences, low dams and channels, etc.) which are important mainly during the first year while the infiltration belt is not yet fully effective. It is also advisable to combine the infiltration belt with a channel connected to the hydrographic network to safely convey excess water from flood rains away from the land protected.

Shrubs play an important role in the infiltration belts: they roughen the soil surface, slow down runoff and support the infiltration of water into the ground. *Elder, woodbine, buckthorn, spindlewood, privet* and *hazel* are usually planted in such belts.

On slopes, infiltration forest belts should follow the direction of the contour lines and the distances between the belts should correspond to the so-called safe width of hillside fields, i.e., the width of noncovered soil surface on which the amount of soil washed away with surface runoff does not exceed the admissible level. The width of such belts is determined on the basis of the condition that all the surface runoff should be transferred to underground runoff at the site of the belt. The total amount of water that has to be infiltrated into the soil under the belt is the sum of the amount of water coming from the upstream area and the amount of water that falls onto the belt itself (Fig. 255).

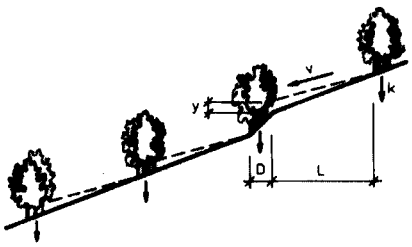


Fig.255 Interruption of surface runoff by infiltration belts.

This relationship (expressed per unit length of the belt) can have the form of the following equation:

$$Dk = vy + Di_s \qquad \text{(m)}, \tag{9.11}$$

where D – width of the belt (m),
k – rate of infiltration into the soil downstream from the belt ($\text{mm}\cdot\text{min}^{-1}$),
v – rate of flow of incoming water (from the area upstream from the belt) ($\text{m}\cdot\text{s}^{-1}$),
y – thickness of the layer of water running down from the upstream area [to be calculated from equation (9.13)] (m),
i_s – precipitation rate ($\text{mm}\cdot\text{min}^{-1}$).

The incoming flow rate (from the area upstream from the belt) can be expressed by the following equation, based on KOSTYAKOV'S (1951) investigations:

$$v = \nu\sqrt{m\varphi i_s L\sqrt{I}} \qquad (\text{m}\cdot\text{s}^{-1}), \tag{9.12}$$

where ν, m – coefficient (Tab. 86),
φ – runoff coefficient,
L – safe width of hillside field (m),
I – gradient of slope.

$$y = \sqrt{\varphi i_s \frac{L}{m\sqrt{I}}} \qquad \text{(m)}. \tag{9.13}$$

Equations (9.12) and (9.13) can be inserted in equation (9.11) to obtain:

$$Dk = \nu\sqrt{m\varphi i_s L\sqrt{I}}\sqrt{\varphi i_s \frac{L}{m\sqrt{I}}} + Di_s,$$

from which the required width of the infiltration belt can be derived:

$$D = \frac{v\varphi i_s}{k - i_s} L \qquad \text{(m)}. \tag{9.14}$$

Table 86

Values of coefficient v (after VESELÝ, 1964)

Gradient of slope %	v	Gradient of slope %	v	Gradient of slope %	v	Gradient of slope %	v
1	1.271	11	1.865	21	2.068	35	2.245
2	1.420	12	1.891	22	2.084	40	2.293
3	1.515	13	1.916	23	2.100	45	2.337
4	1.586	14	1.939	24	2.113	50	2.376
5	1.644	15	1.960	25	2.127	55	2.412
6	1.693	16	1.980	26	2.140	60	2.447
7	1.735	17	1.999	27	2.153	70	2.508
8	1.773	18	2.018	28	2.166	80	2.562
9	1.806	19	2.036	29	2.179	90	2.611
10	1.837	20	2.052	30	2.190	100	2.655

Example 11

The sloping land in Fig. 256 is protected against the loss of soil by a system of infiltration forest belts. The distribution and species structure of the belts do not meet the requirements for erosion control. Belt A, more or less following the slope line, fails to intercept surface runoff and, in addition, unnecessarily divides the land and hinders its cultivation. Belts B and D fail to reach as far as the part of the slope with the greatest gradient which is most severaly affected by water erosion.

Designing a system of infiltration belts

The design of the infiltration forest belts incorporates belts B, C, D and part of belt E. Belt A and the part of belt E which crosses the ridge line are to be removed. Belts B and D are to be extended to the end of the field at the edge of the gully. Belt B is to be extended on the other end towards the ridge line. The effectiveness of the system is assessed on the basis of an analysis of exposure of the steepest part of the slope to water erosion. Lengthwise gradients of the areas between the belts are determined from the lengthwise section (Fig. 257) and the slope length guaranteeing safety of the field against erosion is determined for the given conditions. The calculated slope length is compared with the actual spacing of the belts.

The calculation is performed under the following conditions:

$H = 550$ mm
$i_s = 34.8\ \text{mm}\cdot\text{h}^{-1}$
$\varphi = 0.6$
content of particles 0.001 − 0.1 mm = 37%
content of particles 0.1 − 2 mm = 63%
soil permeability = $2-6\ \text{cm}\cdot\text{h}^{-1}$
admissible amount of soil removed = $4\ \text{t}\cdot\text{ha}^{-1}\cdot\text{year}^{-1}$

The results of the calculation are shown in Tab. 88.

It follows from the results that the proposed system of infiltration forest belts provides adequate protection of the land against erosion. Belt F designed for the northeastern part of the slope (gradient 12.5%) will be located below belt E at a distance corresponding to the safe length of hillside fields whose value, calculated according to different authors, is:

WISCHMEIER − SMITH (1978): $L = 348.71$ m
CABLÍK (1963): $L = 116.66$ m
VESELÝ (1964): $L = 116.66$ m

The distance $L = 116.66$ m was chosen from the results calculated.

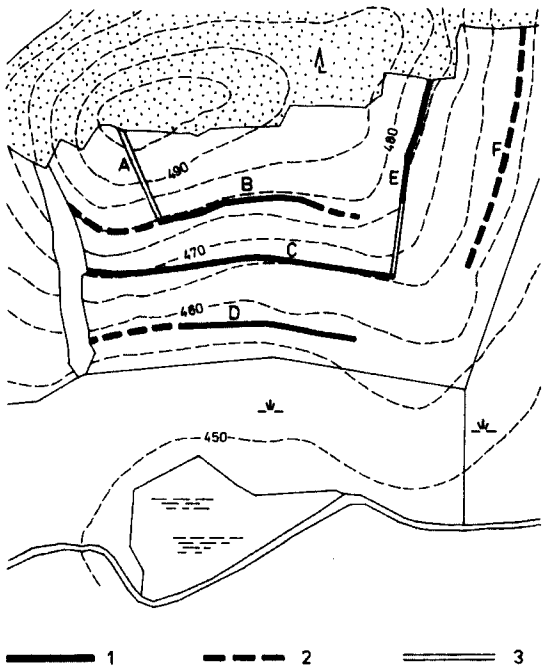


Fig.256 Design and management of an infiltration belt system: 1 – existing belt, satisfactory, 2 – designed elongation of the existing belt, 3 – proposed removal of existing belt.

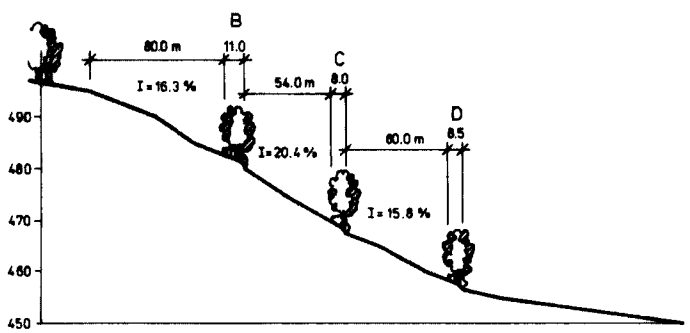


Fig.257 Longitudinal profile through a slope protected against erosion by a system of infiltration belts.

The required width of infiltration forest belts is calculated from the formula:

$$D = \frac{v\varphi i_s}{k - i_s} L ,$$

Table 87

Calculation of the safe length of slope

Part of slope	Gradient of slope %	Safe length of slope L' (m)			Actual length of slope L (m)
		WISCHMEIER-SMITH	CABLÍK	VESELÝ	
above belt B	16.3	123.74	78.34	83.33	80.0
between B and C	20.4	54.79	55.95	59.99	54.0
between C and D	15.8	163.80	82.09	91.66	60.0

where, in addition to the symbols explained above,

v – coefficient from Table 86,

k – 5.43 mm·min^{-1},

i_s – 0.58 mm·min^{-1}.

The results obtained for each belt on the basis of this formula are shown in Tab. 88.

Table 88

Width of the infiltration forest belts

Belt	B	C	D	E	F
D (m)	11.40	8.02	8.50	8.17	15.79

Designing the optimum species composition and spatial arrangement

The unsuitable shrub and tree stands in the existing infiltration forest belts will be converted to a more effective structure by removing the undesirable species (e.g., fruit trees) and by thinning the larger shrub groups. More effective species, adequate to the site conditions and to the structure of the surrounding stands, will be planted in the free places cleared by removing the undesirable shrubs and trees. Simultaneously with the change of species composition, the belt will be extended by new planting along the upper boundary of the belt to attain a width determined by calculation.

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Subject Index

A

Abrasion 216, 221, 372, 373
Accumulation 25, 27, 58, 89
Afforestation 300, 323, 326, 327, 330, 334, 341, 344–348
– conservation 342
Agriculture 16, 17, 18, 21, 22, 23, 33, 41, 102, 103
Alignment 186, 188, 231
Anti-erosion grassing
Arable land 31, 33, 37, 65, 66, 101, 103, 135
Arch 261, 262, 279–281
Apron 228–230, 232, 234, 236, 237, 242, 259

B

Bed 195, 224, 230, 231, 236, 244
Bed load 153, 155, 158, 166, 173, 189, 190, 197
Bed-load discharge 173, 174, 176–178
Bed slope 183, 189, 195–199, 228, 231, 258, 296
– equilibrium 190, 258
– stable 183, 190, 191, 192, 194, 224, 296
Belt
– forest shelter belt 378, 382
– infiltration belt 378, 384, 385
– rock 226, 227
– stabilisation belt 198, 224, 225, 226, 236, 237
– stabilisation, concrete belt 226
– stabilisation, stone-filled gabion belt 226
– stabilisation, stonework belt 226
– windbreak, impermeable belt 378, 379, 381
– windbreak, permeable belt 378, 380, 381
– wooden belt 226, 227
Bottom 195, 224, 225, 226, 229, 230
– cover layer 158, 160, 162, 197
– surface layer 158, 160, 162, 166, 167, 176, 190, 197

C

Channel 148, 171, 173, 185, 186, 195, 224, 225, 227, 232, 242, 312, 349–351, 354, 363–367, 369, 384
Chute 227, 244, 312
– area 245–251
– bank 245
– crest 245, 246, 248, 249
– downstream floor 244, 245, 246, 247, 250, 251
– rock 244–300
– surface 244, 247, 250, 251
Catchment 24
Clayonage 346
Clipping 338
Conservation 22
– tillage 112, 115
Contour cultivation 111
Contour roads 110
Corrosion 36
Crop rotation 75, 77
Cropland 105
Crops 113
Cross-section 198, 199, 239, 248
– area 200, 238
– design 183, 195, 198
– dimension 199
– shape 198, 200
Crown closure 334, 338, 359
Cutting 213, 221, 222, 223, 296, 322, 327, 365, 366

D

Dam 195, 295, 296, 299, 384
– arch abutment 261, 262, 280, 282–284
– arch dam 260, 262, 279
– body 256–260, 264, 269, 274, 275
– cantilever overturning stability 267, 269
– cantilever sliding stability 268, 269
– cantilever-type gravity dam 259, 267, 269
– check dam 190, 230, 233, 235, 237, 255–257
– concrete dam 233, 260
– consolidation dam 255, 257, 258, 300
– construction 256, 257, 259, 263, 264
– gravel gabion dam 263
– masonry dam 259

- monolithic dam 260, 274, 276
- prefabricated reinforced-concrete dam 262
- retention dam 255, 256
- static calculation 264, 267, 274, 279
- woody dam 263
Deflation 36
Delimitation 102, 104
Depth 51, 129, 131
- conjugate 237–239, 250, 251
- sowing 330
Digging 117
Discharge 148, 176, 200, 211, 238, 239
- design 183, 184, 189, 198–200, 205, 248
Ditch 130, 132, 312, 313
- drain 290
- interception 40, 294, 312
Drag
- coefficient 168, 169
Drainage 19, 323, 341, 348
- basin 42, 106
- systematic 313, 363
Duststorm 36

E

Edaphon 16
Energy
- dissipation 236, 237, 244, 247, 251
- kinetic energy 189, 229, 237, 242, 244, 246, 247
- line slope 174, 175, 176, 200, 238, 249
Erosion
- anthropogenous 25
- areal 30
- furrow 31, 113, 291, 347
- glacial 29
- ground 29
- manifestation 30, 36
- rill 21, 290
- scouring 33
- sheet 31, 290, 340, 345
- snow 29
- stream 30, 35
- types 25
- water 25, 81, 290
- wind 28, 36, 89, 340, 378, 381, 382

F

Factor 39
- erosion control measures 118
- erosion effect 72
- field shape 106, 107
- field size 108
- slope gradient 42, 72, 73, 74
- slope length soil erodibility 43, 73
- vegetation cover 45, 75
Famine 21
Farming 17, 80, 111
Fascine 208
- bundle 213, 217
- gravel pole 214, 215
- pole 213, 214
Fence
- lath 217, 230, 294, 295, 346, 363, 367, 384
- vegetated lath 223, 230
- wicker 205, 208, 215, 344–346, 374
Field
- position 72, 106
- length 73, 106, 108
Flow
- nonuniform 237, 238, 248, 249
- subcritical 237, 238, 244, 248
- supercritical 198, 237–39, 242, 244, 247, 250, 251
- uniform 199, 238, 248, 249
Forest 17
- commercial 318
- composite forest with standards 319
- coppice 319
- high 319
- microclimatic and hygienic function 322
- nursery 323, 324, 330
- protection 318
- recreational function 323
- soil conservation function 323
- soil improvement function 323
- special purpose 318
- water-management function 320
Furrows 113, 290, 294, 328, 346

G

Galleries 310, 313
Geotextile 347
Grass 61, 76, 77
- carpet 124
Grassing 212, 223, 296, 341
Grassland 76, 105
Guard strip 122

Gully 37, 290, 291, 345–347
- afforestation 295, 296
- brush plug 294
- causes of gully erosion 33, 290, 293
- channel 293–295, 299
- forms 291
- headcut 292, 295, 298, 346
- headcut control 294
- side slope securing 296
- stabilisation 293, 295, 299

H

Hillside 103, 105
Hole
Humus 17, 45
Hydraulic 132
- calculation 237, 247
- jump, adjoining 237
- jump, submerged 237
- jump, unsubmerged 237, 239
- uplift pressure 267
Hydrodynamic
- lift force 166, 167
- pressure force 166, 167
Hydrological characteristic 41, 47, 49–66, 131
Hydroseeding 341
Hydrostatic pressure 264, 267, 268, 281, 302

I

Inclination 43, 107
Infiltration 51, 53, 320, 322, 384, 385
Interception 45, 321, 322, 381

L

Land 16, 18, 22
Landslide 258, 300–305, 308, 310, 314, 315, 340, 345, 347
- causes 301
- control methods 308
- management 308
Lining 198, 205, 216, 221, 222, 230, 360, 361, 367, 371
- bottom 196, 197, 205, 206
- combined 208, 221
- nonvegetative 217
- vegetative 208, 211, 222
Loam 302, 306
Lowlands 47, 359, 367, 381

M

Modelling erosion processes 94, 99
Moisture 41, 51, 91, 342, 378, 381, 382, 384
Monoculture 17, 45, 72, 118
Mulching 342

N

Nappe 232
- overflowing 232, 236–238, 242, 243
Nutrients 16, 46, 322, 382

P

Path 109
Pavement
- dry 206
- in cement mortar 206, 207
- large boulders 207
- local stone 207
- quarry stone 205, 207
- stone 218
Perimeter
- wetted 128, 202, 227, 239, 354
Plant 325, 329, 330, 331, 334, 341, 342, 344, 346, 349, 351, 356, 365–367, 369
- ball 333
- container 324–326
Planting 325, 326, 330, 333, 334, 337, 338, 344, 366, 367, 369, 380
- bunch 331
- cordon 345
- corner 333
- deep 331
- density 334
- double 331
- elevated 331
- furrow 333
- hill 331, 333, 348, 366
- mound 331, 333, 348, 366
- notch 333, 348, 383
- pit 331, 348
- season 334
- single 331

- of willow cuttings 345
Pollution 17, 37
Propagation 327
- vegetative 327
Protection of cultures 22, 101, 111
Pulling 338, 339
Pruning 338–340

R

Rain 30, 40, 31
- design 48, 55, 64
- factor 69, 70, 94
- intensity 48
Reservoir 371–373
Revetment 185, 198, 199, 246
Riparian stands
- association 360, 364
- aquatic plants 351, 373
- channel slope control 349
- influence on the flow rate 351
- mission 357, 364, 369
- overgrowing and silting control 350
- roughness coefficient 353, 354, 356, 357
- self-cleaning capacity 351
- spatial structure 360
- species composition 357
Riprap 196, 199, 205, 206, 208, 213, 218, 224, 227, 370, 373
- vegetated 221, 222, 367, 374
Roots 209, 296, 298, 303, 319, 324–327, 331–334, 339, 348, 350, 351, 357, 360–363
Roughness
- bottom 159, 175, 190, 197, 239
- channel 130, 189, 200
- coefficient 200, 202, 212, 353, 354, 356, 357
- lining 200, 237
- sediment 17, 45, 200
Runoff 41, 47, 148
- area 50, 55, 57
- coefficient 56, 57, 60
- curves 57, 66
- maximum 56, 64
- surface 47, 61, 64, 67, 290, 293, 294, 322, 343, 371, 384

S

Scour 224, 225, 227, 247, 257, 258, 259, 300, 342
- bank 183, 186, 208, 215, 218, 220, 322, 365
Sediment 148, 153, 198, 202, 255–258, 267, 365
- effective grain 161, 184, 196, 202
- grain density 154
- grain mass 154
- grain shape 153, 154, 155, 170
- grain-shape categories 156
- mixture 156, 158, 159, 189, 259
- mixture properties 158
- movement 158, 166, 172, 173, 174, 190
- properties 153
- runoff 178
- sampling 160
- size 153, 154
Seed 324–330, 340–342, 344, 349, 383
Seeding 212, 324, 338, 340, 341
- broadcasting 330
- pocket 330
- row 325, 330
- spot 330
Seedling 324, 325, 326, 330
Sill 195, 227–230, 236, 237, 242, 296, 300
Slippage 301, 308, 310
Snow guard 30
Sodding 205, 212, 296, 341
Soil
- coarse 45
- conditions 45, 75, 291
- degrading 33
- depth 16, 45, 51
- erosion 25, 37, 475
- fertility 17, 33, 118, 336
- improving species 118
- irrigation 17, 24
- loss 30, 37, 71, 93, 94
- particles 45
- permeability 45, 70, 86
- profile 18, 45, 102, 118, 309
- resistance 45, 75, 111
- skeleton 33
- surface 25, 81, 118, 312, 382
- texture 70, 87, 89
- topsoil 89, 90
Sorption capacity 45, 118
Sowing 205, 212, 324, 325, 329, 330, 341
Stand
- mixed 331, 336, 339
- riparian 326, 349, 350, 351, 357, 360, 361–365, 367, 369–372
- soil-conservation 118, 124, 340

Stilling basin 228, 244, 247, 251, 259
Stone
- filled gabion 205, 219
- filled mattresses 220, 222
- packing 205, 208, 218, 222, 229, 230, 236, 237, 239, 246, 247, 250, 251, 296, 299, 346, 347, 363, 367, 370, 373, 374
- pitching 205, 206
- vegetated packing 222
Stress
- allowable, bending tensile 274, 275
- allowable, compressive 281
- bending, tensile 263
- critical shear stress 167, 172, 173, 190, 194, 250
- shear 172, 175, 194, 250, 354
- tangential 84, 129, 133, 312
- tensile 262, 267, 269
Strip
- cropping 78, 116, 122
- cultivation 116

T

Tending 330, 331, 336, 337, 338, 340, 342, 382, 384
- of new growth 338
- of stands in pole stage 338, 339
- of stands in small pole stage 338, 339
- of thickets 338, 339
Terrace 95, 125, 136
- channel 126, 128, 129
- earth 137
- fence 140
- rock 141
Terracing 72, 78, 125, 136, 341, 343
Terrain regulation 125
Thermal regime 39
Tilling 19, 102, 114
Topography 42, 102
Torrent 148, 172, 173, 182, 198–200, 215, 224, 228–230, 234, 237, 244, 255–257
Transplanting 324–326, 334
Transport 25, 37, 56, 148, 156, 178

V

Vegetation 45, 75, 95, 105, 115, 118, 318, 320, 322, 323, 340, 341, 350, 354, 372–374
- cover 45, 76, 96, 290, 291, 303, 314, 340, 342
Velocity 81
- average channel 126, 197, 200, 205, 248, 356
- bed 168, 174, 177, 190, 191
- coefficient 82, 83, 85, 126, 174, 200, 202, 238
- critical, average channel 171, 173, 190, 192, 193, 197
- critical, bed 167, 171, 173, 174, 190, 192
- shear 174, 190

W

Wall
- log 218, 220, 246, 300, 374
- retaining 206, 220, 221, 228, 300, 315
- palisade 346
- vegetated 222, 346, 347
Water
- ways 41, 126, 130
Weir 195, 198, 227, 230–237, 242, 257, 300
- apron 232, 234, 236, 237
- body 232–237
- concrete 232, 233
- combined 235
- downstream floor 236
- hydraulic calculation 237
- masonry 232, 235
- prefabricated 232, 234
- stilling basin 236, 237, 242, 244
- stone-filled gabion 232, 235
- wooden 232, 234
Willow 208, 209, 210, 211, 221, 222, 223, 327, 345, 347, 357, 359, 360, 365, 373
- brushwood covering 208, 216, 222
- osier mats 217
- shrubby 208, 210, 211
Wind
- speed 90, 91, 378, 379, 380, 381, 382
Woodland 124

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